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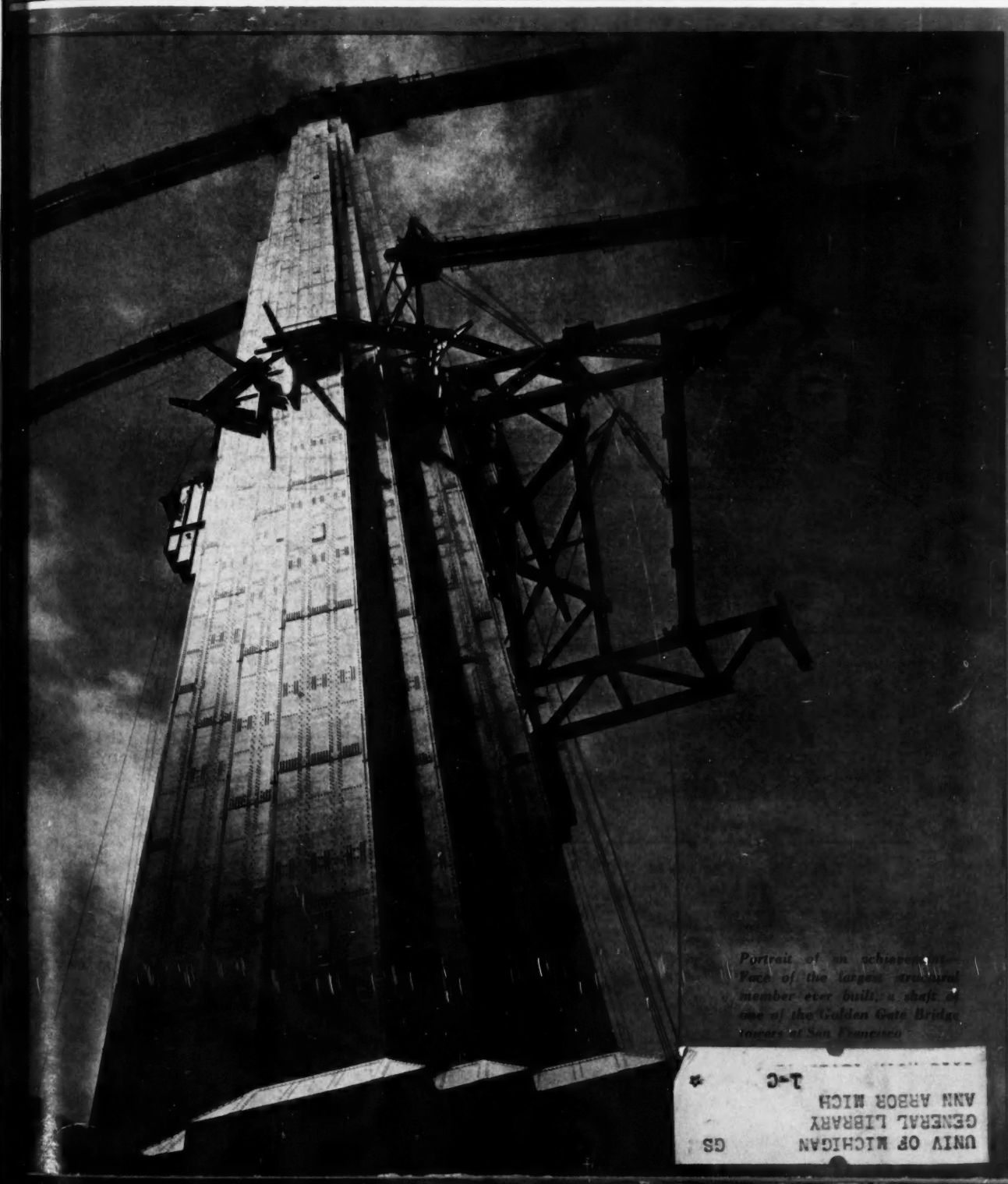

Portrait of an achievement—Piece of the largest structural member ever built; a shaft of one of the Golden Gate Bridge towers at San Francisco

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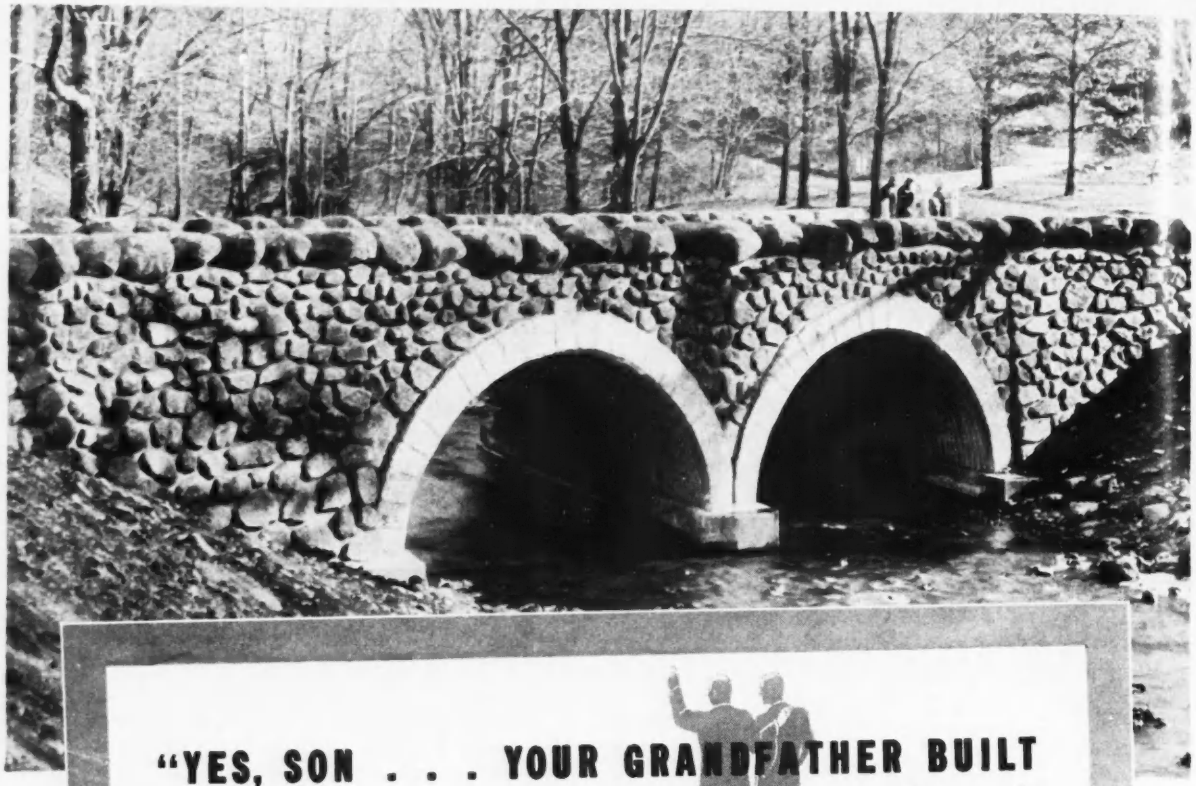

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A MOMENT WITH THE PUBLISHERS

To FIT MEN *for* NEW JOBS

FROM many sources we learn today of a shortage in skilled labor. The tide of recovery has caught industry off its guard. Orders cannot be filled; qualified and competent workers cannot be found for regular jobs at good wages. Yet we still hear of millions on relief and unemployed. In one quarter the shortage is emphasized; in another the remaining unemployment is played up. The thoughtful try to see both and to appraise their significance.

It is true that we always have had and always shall have a substantial number of unemployed. Some of them are unemployed from choice but more actually are unable to find jobs, i.e., buyers for their services. Probably we have right now more unemployed than we normally should, yet it is quite possible that whatever unemployment there may be is the more impressive because it has been so effectively dramatized.

IN more normal times men out of work make no parade of their unemployment. But during the depression we tried, very properly, to remove any stigma from unemployment as such, in order to preserve the morale of those who, although anxious to work, were unable to find jobs. Human nature being what it is, many who are idle from lack of ambition or competence have merged themselves with the deserving unemployed in order to share in the relief provided by the community for the latter. It is well-nigh impossible, therefore, to get an appraisal of unemployment volume that takes this moral factor into account. The task is not made easier by a political campaign that makes an issue of unemployment and its relief.

But however much we may discount any unemployment figures, it is evident that we have a substantial number of men who can find no jobs, while at the same time, we have many jobs that can find no men qualified and competent to fill them.

All this gives rise to many questions that must engage the attention of those who would solve the riddle of unemployment. What does it mean, for example, with respect to the so-called technological unemployment? What does it suggest as to adult education? Does it not indicate the need for approaching the employment problem as one of *re*employment rather than as one of *un*employment? Is it not clear that we face an increasing necessity for training men to new skills so that they may find places in the new industries and callings?

With the technical progress of industry there is bound to be constant change in the relative demand for various types of skill. The task of seeing that men are enabled to meet those changes is as much the problem of organized industry as it is of the individual or of the socially minded. If industry is to reap its richest rewards it must find places somewhere in its ranks for the largest possible number of the people. It needs their productivity; it needs their purchasing power: the two are inseparable.

THIS does not imply that industry, which carries the responsibility for efficient production and stakes its existence on that efficiency, should be required to carry men on its payrolls whether or not they are qualified and competent. It does mean that, in cooperating with the community to establish agencies for the reconditioning of men so that they can pass over from obsolete jobs to new jobs, industry would be working for its own advantage as much as for that of society as a whole.

It is far better to keep such a check on unemployment through participation in a joint agency of business and government, designed to fit men for new work before writing them off as social liabilities, than it is to be short of the skilled men we need to maintain our productivity while we carry on our backs by welfare taxation many who are idle only because they lack the skills for which there is demand.

Our problem is one of *re*employment, not *un*employment. No one profits from unemployment, however effectively we may support its victims.

Willard Chevalier

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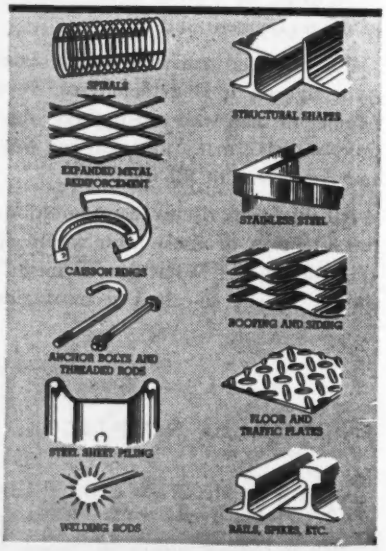

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ENGINEERING NEWS-RECORD

October 8, 1936

Designing Bridge Towers 700-Ft. High

Bold rigid frame structures which support 4,200-ft. span Golden Gate suspension bridge subjected to rigorous analysis culminating in 33 simultaneous equations whose development was aided by ingenious application of Williot diagram procedure

By Clifford E. Paine
Strauss & Paine, Inc., Consulting Engineers, San Francisco

THE TOWERS of the Golden Gate suspension bridge rank high among the notable achievements of the structural art. Bold beyond comparison with any other bridge tower by virtue of the omission of diagonal bracing between the shafts above the roadway and because of their skyscraper height and the tremendous forces, which include wind and earthquake, that they must resist, their design called for unusual technical skill, rigorous analysis and tedious computation. In concise language and precise terms, Mr. Paine outlines the development of the design of these premier rigid frames upon whose stability and strength the safety of the world's longest suspension bridge rests. The towers were analyzed as complete units instead of as an assembly of connected parts, and the care with which every load and strain effect was investigated and incorporated in one of the 33 simultaneous equations necessary for stress determinations is noteworthy. To secure these equations it was necessary to use, in addition to the moment equations, a series of strain equations obtained by ingenious use of Williot diagrams, a procedure that is believed to be an original contribution with which the design of the towers can be credited. (See also ENR, April 26, 1934, p. 534.) The article warrants the close attention of structural engineers and of all others who desire an appreciation of modern structural analysis procedure—EDITOR.

THE STEEL TOWERS supporting the cables of the Golden Gate Bridge at San Francisco, obviously notable because of their size, also command attention because of the unusually close correlation of architectural and structural requirements that characterized their design. Developed as rigid frames made up of two shafts and cross bracing without diagonals in the panels that comprise the upper 500 ft. of their 700-ft. height, the towers each contain 22,200 tons of steel, about 10 per cent more than was used in the George Washington Bridge towers at New York. The main section of each shaft or leg of the towers has an area of 12,391 sq.in. as compared with only 4,500 sq.in. in the Philadelphia-Camden bridge towers of similar makeup and of 5,000 sq.in. in the eight columns that comprise one leg of the George Washington Bridge towers. The Golden Gate tower shafts are thus by far the largest structural steel members ever assembled.

The towers are 690 ft. 4 in. high from the piers to the base of the saddle castings, and rise 746 ft. above the water to the intersection of the main and the side span cable tangents. The center lines of the tower shafts are 90 ft. apart, the distance selected as the desired spacing between centers of cables. This spacing and the necessary width of the tower shafts is just sufficient to provide clearance for the 60 ft. roadway through the towers at deck level.

Supporting a 4,200-ft. center span and two 1,125-ft. side spans through two 36½-in. diameter cables, the towers are designed to resist forces of great magnitude caused by dead and live loads, wind and earthquakes. This fact, coupled with the special indeterminate nature of the structures made the design problem a difficult one. Among the contributions to structural practice that came out of the design is a novel application of the Williot diagram, by which strain equations can be written directly without first knowing the stresses that cause the strains. Erection of the towers was completed in July, 1935, cables were completed in September, 1936, and work is now underway on the erection of stiffening truss and floor steel.

The general features of tower design on the Golden Gate Bridge were developed concurrently with the architectural treatment, to insure a satisfactory architectural effect without undue cost or structural sacrifice. The architectural treatment is in general modernistic and involves the exclusion of applied decoration and unnecessary elements. Where ornament was desirable, either to enrich

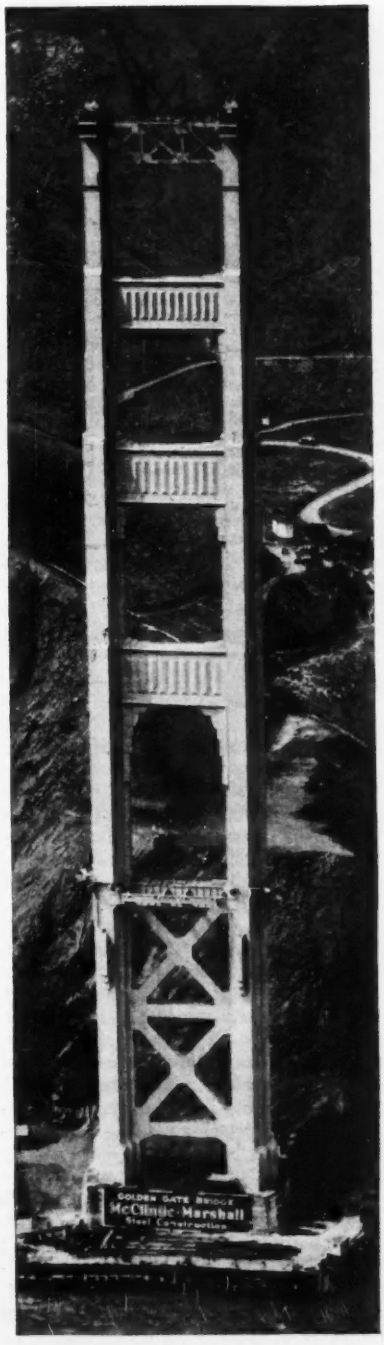


FIG. 1—RISING nearly 700 ft. from pier to saddle casting, the Golden Gate Bridge towers make architectural as well as structural history.

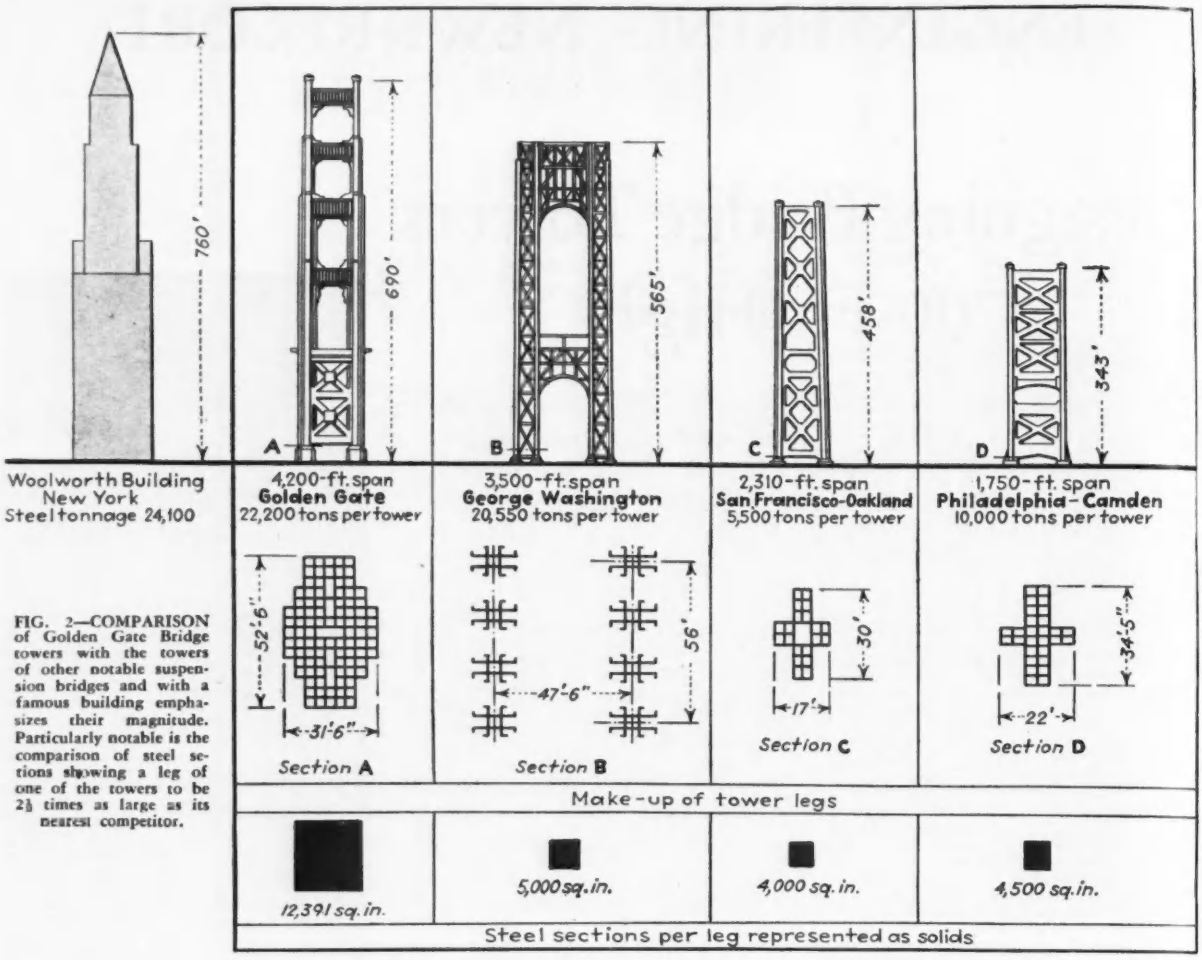


FIG. 2—COMPARISON of Golden Gate Bridge towers with the towers of other notable suspension bridges and with a famous building emphasizes their magnitude. Particularly notable is the comparison of steel sections showing a leg of one of the towers to be 2½ times as large as its nearest competitor.

certain surfaces or to diversify surfaces unlikely to be perfectly executed over large areas, a simple vertical fluting was used consisting of intersecting plane facets which could be carried out readily in both concrete and steel. Since an ornamental elaboration of minor details, such as hand rails, lighting units, etc., is likely to be inharmonious in a structure predominantly engineering and industrial in character, these features were handled in the spirit of the large design, using principally structural steel shapes.

Where great heights are involved, the common architectural practice of enlarging the scale of details as their distance from the ground increases, tends to emphasize the detail as such at the expense of the whole mass. The opposite principle was employed on towers of the Golden Gate Bridge; the scale of details on the towers decreases with the height.

The main shafts of the towers are built up of plates and angles arranged in cellular form (Fig. 3), using sections 3½ ft. square. At the base in the longitudinal direction of the bridge, each shaft is 15 cells wide, and in the transverse direction, 9 cells wide. The base slab on which each shaft stands is a 33x54-ft. steel plate 5 in. thick, made up of 19 separate pieces. As the shafts progress from the base toward the top, the section is decreased by successive omissions of outside cells, until at the top the total number of cells is only 21, arranged in the form of a rectangle composed of 3 cells in the transverse direction and 7 cells in the longitudinal direction.

Below the bridge floor level, transverse bracing between the two shafts is provided in the form of two panels of diagonals. Above the floor level, however, transverse bracing is confined to four portal struts. All bracing members have four webs which are spaced to permit gusset connections direct to web plates of tower shafts; thus the width of bracing members is approximately the width of three cells, or 10½ ft. The portal struts are cased in steel plate enclosures, detailed to carry out the architectural treatment.

Factors that influenced design

The principal architectural influence on the structural make-up of the towers is evident in the portal strut bracing. This form of bracing was found to be efficient, and did not involve any structural difficulties; the spacing of the portal struts was arbitrarily chosen to give the best appearance.

The dead load of the towers is 22,260,000 lb. per shaft. The dead load concentration from the cables on the saddles amounts to 52,000,000 lb. and the live load and temperature concentration to 9,438,000 lb. per shaft. The towers, cables and stiffening trusses were designed for a live load of 4,000 lb. per lineal foot of any length and position. In the case of the center span stiffening trusses, for combination of live load and wind loads of 30 lb. per square foot, the live load was taken at 2,000 lb. per lineal foot over the entire center span plus 2,000 lb. per lineal foot of any length and position.

Transverse wind forces applied to the tower were computed on the basis of 30 lb. per sq.ft. on twice the vertical projection of trusses and cables plus 50 lb. per sq.ft. on twice the vertical projection of the tower. For longitudinal wind forces computations were on the basis of 50 lb. per sq.ft. on the vertical projection of the tower plus 30 lb. per sq.ft. on one-half the vertical projection of each web member of stiffening trusses and of each floorbeam and knee brace. Temperature stresses were figured for an as-

sumed rise or fall in temperature of 40 deg. F.

At the outset it was decided that the tower shafts were to be of cellular construction formed of 42x$\frac{7}{8}$-in. web plates and 8x8x$\frac{5}{8}$-in. angles. Silicon steel is used from the base up to the roadway and in the top two panels; the two center panels utilize carbon steel. Whenever unit stresses in silicon steel were found to be in excess of 20,000 lb. per sq.in., the thickness of the plates was increased to $1\frac{1}{8}$ in. This occurs, for the most part, in the outer cells where bending stresses are of more consequence. Also it was decided at the outset that the l/r ratio of the tower shafts should not exceed 60. The establishment of these proportions pretty well determined the number and arrangement of cells.

The carbon steel specifications required tensile strength of 60,000 to 70,000 lb. per sq.in. and a minimum yield point of 36,000 lb. per sq.in.; for the silicon steel a tensile strength of 80,000 to 95,000 lb. per sq.in. and a minimum yield point of 45,000 lb. per sq.in. were specified.

The allowable unit stresses adopted were as follows:

Direct stress in shafts from dead and live loads and temperature changes—
Carbon steel 14,000 lb. per sq.in.
Silicon steel 18,000 lb. per sq.in.
Combined direct and bending stresses in shafts—
Carbon steel 18,000 lb. per sq.in.
Silicon steel $\frac{7}{8}$-in. plates 20,000 lb. per sq.in.
$1\frac{1}{8}$-in. plates 23,000 lb. per sq.in.
maximum 24,000 lb. per sq.in.

Longitudinal stresses

The towers are deflected 6 in. toward the shore in their dead load, normal temperature, position. From changes in the sags of the side span cables and from changes in length of the side span cables due to temperature and loading, the tower tops may move to positions 18 in. channelward or 22 in. shoreward from vertical lines through the centers of the tower bases. These movements, determined by the most unfavorable positions of the full live load of 4,000 lb. per lineal foot of bridge and a maximum temperature change of 40 deg. F. up or down from normal, are smaller than on some shorter spans because of the favorable ratio of side span to main span, the low proportion of live load to dead load, and the moderate temperature change. Despite the limitation of the longitudinal radius of gyration to 1/60 of the height of the tower, the maximum unit bending stress from a deflection of 22 in. is only 6,000 lb. per sq.in. Under such deflection the bending moment in the shaft at the base is slightly over 3 billion in.-lb.

The analysis of stresses is simple and conventional but the important effect of the eccentricity of the direct loads in reducing, by more than one-half, the amount of horizontal force from the cables required to produce the desired deflection, necessitated several sets of computations before the assumed deflection curve agreed with the computed curve.

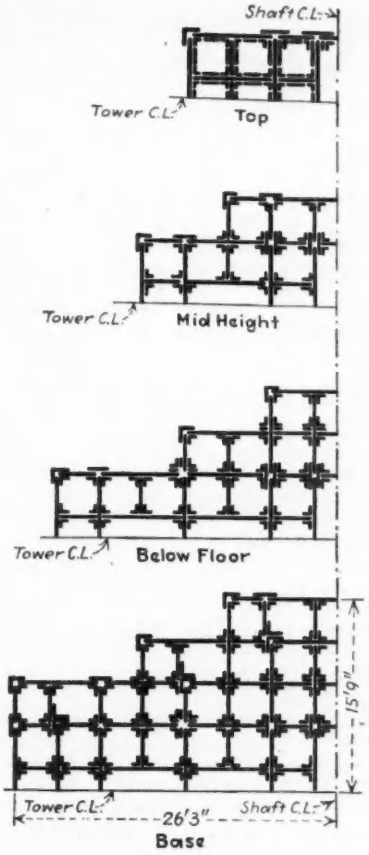


FIG. 3—CELLULAR MAKEUP of tower shafts whose sections set a new high record for steel assemblages; at the base each Golden Gate tower shaft contains 12,591 sq. in. The Camden bridge towers, of comparable cellular makeup, next in size, contain only 4,500 sq.in. per leg. The eight columns comprising one leg of a George Washington Bridge tower have a total base section of about 5,000 sq.in., but are entirely of silicon steel figured on a high unit stress basis.

Several cases of longitudinal wind combined with longitudinal tower deflection were also studied. However, except for a short distance near the middle of the tower, the longitudinal wind forces did not affect the design of the tower.

Transverse wind stresses

The transverse wind load per shaft amounts to 3,149,000 lb., of which 30 per cent is from the cable, 26 per cent from the laterals of the suspended spans, and 44 per cent is the wind on the tower itself. Since the history of the design of the towers is practically the history of the determination of the transverse wind stresses, some description of the various methods used to analyze the wind stresses is pertinent.

Determination of the superimposed loads, the longitudinal tower deflections, the maximum slenderness ratio, the unit working stresses and an outline of the structure completed the preparatory steps. The direct stresses from dead load and live load were then tabulated and a guess made at the amount of the unit bending stresses from longitudinal movements. The wind stresses were computed on the basis of a point of contraflexure at the center of each unbraced panel. This information was sufficient to permit an approximate determination of the various sections of the shafts, struts and other web members.

Using the tentative sections selected, the longitudinal bending moments were computed and the sections revised to correct the errors made in guessing the amount of the unit stresses from longitudinal bending.

To facilitate investigation of the tentative design under transverse forces, the tower was cut at the floor, and the upper part of the tower figured as an independent structure, consisting of a multiple-story portal frame. The solution assumed identical deflections and changes in slope for the two tower shafts. The fully-braced lower section of the tower was considered as a simple truss. This analysis gave somewhat different results than the first determination of wind stresses, and the sections were again revised.

The methods used so far were very crude in their determination of the stresses. For a more accurate analysis conventional slope deflection methods were next employed. It was assumed that the direct wind stresses previously found were correct; that the struts had no depth (Fig. 4); and that the base of the tower was free to rotate. A conventional Williot diagram was drawn and the usual slope deflection equations were set up and solved to determine the moments in the towers. Direct wind stresses resulting from this solution were substituted for those assumed and a new solution carried out. After several attempts the calculated direct stresses agreed fairly well with those assumed. Sections were revised, longitudinal moments recalculated, sections again revised and the transverse wind stresses again computed. This completed what may well be termed the "Preliminary Design."

Refinement of preliminary design

At this stage a critical study of the tower design was made. Some objectionable features were embodied in the preliminary design. Webs, for example, in the longitudinal and transverse faces of the shafts varied in thickness from $\frac{7}{8}$ to 3 in. The splices for webs of 3 in. thickness were long and the riveting burdensome. The gussets for the web members below the floor were very long and the bending stresses in the gussets became large when the intersections of the working lines of the horizontal and diagonal web members were placed on

the center of the shaft. Some of the portal struts had short heavy members and the secondary stresses in these members certainly would be high.

The basic assumptions used thus far in the stress determination were far from satisfactory. Obviously an assumption of fixity for the bases of the shafts was more nearly correct than the previous assumption of a free base. The vertical dimension of the struts would have a considerable effect and must be considered. Also the shear carried by the shafts would have to be considered, to make any accurate determination of the direct stresses in the diagonals below the floor.

A survey of the preliminary design indicated that if proper consideration were given to the depth of the struts the main web plates in the shafts could be kept at ⅞-in. thickness, although in certain places they would have to be made of silicon steel, and in a few instances reinforcing plates between the angles would have to be added. This would produce a much cleaner design. There was no reason why the intersection of the working lines of the web members needed to be on the centerlines of the shafts if account were taken of the resulting stresses from the eccentricity. Also, the strut sections and their panel lengths were revised in an attempt to keep the anticipated combination of primary and secondary unit stresses within permissible limits.

The errors of assumptions and analysis used in the determination of the stresses in the preliminary design were next to be eliminated. A correction of the previous calculations showed that the assumption of fixed bases gave moments of considerable magnitude in the shafts below the floor. The shears accompanying these moments were of such magnitude as to affect considerably the stresses in the web members. Also the appreciable moments resulting from eccentric connections of the web members below the floor had to be included in the calculations. Thus the part of the structure below the floor changed from a very simple truss to a statically indeterminate structure. Study of this perplexing problem led to a clear-cut analysis.

Unique use of Williot diagram

In all previous work with slope deflections the primary stresses had been obtained first without regard to the bending stresses, and then a Williot diagram was drawn using the strains derived from the primary stresses to obtain the deflection angles of these web members. It was now realized that the Williot diagram was the missing key to a satisfactory, direct solution. A Williot diagram was sketched without knowing the stresses but from which it would be possible to evaluate the deflections in terms of the various unknown stresses. The unknown stresses could be readily expressed algebraically in terms of the known loads and the unknowns of the slope deflection formulas. This procedure took care of the major difficulties. Placing the angle of rotation of the shaft at the base equal to zero provided the necessary fixity of the base of the shaft, and the inclusion of the eccentric moment from the diagonals below the floor in the summation of moments at the various points took care of those stray moments. Thus the lower part of the structure was adequately analyzed although the solution had to await the necessary equations for the portal-braced, upper portion of the tower.

Taking account of strut depth

Transferring attention now to the upper portion of the tower, a possibility of treating the struts in a manner similar to that developed for the part below the floor was considered. However, it was soon evident that the number of equations would become unwieldy. Also it was noted that the secondary moments in the strut members were far too small to affect the moments in the shafts more than a fraction of one per cent. Apparently the effect of secondary stresses in the struts could be handled adequately without combining this study with the stresses in the structure as a whole.

This decision left the introduction of strut-depth into the analysis as the major remaining problem in this part of the structure. It was found that by using a special equation for the deflection angle of the shaft between the chords of a strut, a satisfactory analysis could be made. This method, described below, is believed to be unique.

First, assume that the end posts of a strut—the shafts between the chords of the strut—are restained so that their axes remain vertical while the leeward end of the strut is permitted to deflect upward under the action of an upward force equal to the shear in the strut (Fig. 5). If now the strut, including its end posts, is rotated until the leeward end is again at its initial elevation, the end posts will have rotated through an angle equal to the deflection of the strut divided by the distance between the posts. The deflection angle, due to the distortions within the strut as above determined, is increased by the change due to the shortening of the leeward post and the lengthening of the windward post. This added deflection angle is equal to the difference in elevation of the two ends of the struts divided by the distance between the posts.

The shear in the struts was expressed in terms of the known loads and the unknown deflection angles and rotation angles of the shafts. To eliminate the necessity for repeated solutions the elongations and shortenings of the shafts from direct wind loads were also expressed in terms of the known loads and unknown angles.

This method contains an approximation in the assumption that the end post of the strut does not change length. It was assumed, in computing that part of the deflection angle of the shaft between strut chords due to changes in length of the shafts, that the shaft changes in length were from the base of the shaft to the center of the strut. This assumption produced only a small error in the structure as a whole. Account was taken of the changes in length of the shaft between chords when studying the secondary stresses in the struts.

The analysis of the towers was finally concluded with a set of 33 simultaneous equations. Since most of these equations did not contain more than half a dozen unknowns, the solution was not difficult. Having the necessary distortion angles it was possible to compute in the customary manner all of the required stresses from transverse wind.

The moments in the shafts were, of course, appreciably smaller than in the previous calculations. This resulted in the anticipated savings in material, and, together with a few changes to silicon steel, permitted the web plates to be of uniform thickness except in a few places where the stresses were sufficiently high to require ⅛-in. plates as a precaution against web buckling. At the base of the shafts the plates were thickened to ⅛-in. to resist corrosion. The transverse deflection of the tower by this analysis was shown to be 12½ in. instead of 16 in. as obtained by the analysis used in the preliminary design.

The 33 equations in detail

The development of the equations is outlined to illustrate the procedure. There are 5 equations of the "bent" type such as:

$$M_{qt} + M_{tq} = -380W + D\ (380R_{qt})$$

where W is the external wind shear and D the direct dead and live load stress in the shaft.

There are 13 equations of the usual kind equating the summation of the moments at a point to zero. The summation of the moments at points k, o, and q, (Fig. 4), include the moments due to eccentric web connections.

There are 3 equations for the strains in members *kt*, *ot*, and *qt*. The development of these equations will be outlined later.

There are 4 equations for the deflection angles of shaft members between the chords of the 4 struts. These take the form

$$R_{bs} = -\left[\frac{\Delta_{1t}}{540} + \frac{0.000{,}000{,}724{,}034\ T_1}{1080}\right]$$

in which Δ_{1t} is the change in length of the shaft from the center of strut 1 to the base of the tower; the factor 0.000,000,724,034 is the vertical deflection, from a unit load, of the strut as a beam fixed at each end; and T_1 is the vertical shear carried by the strut. See Fig. 5.

There are 8 equations for the deflec-

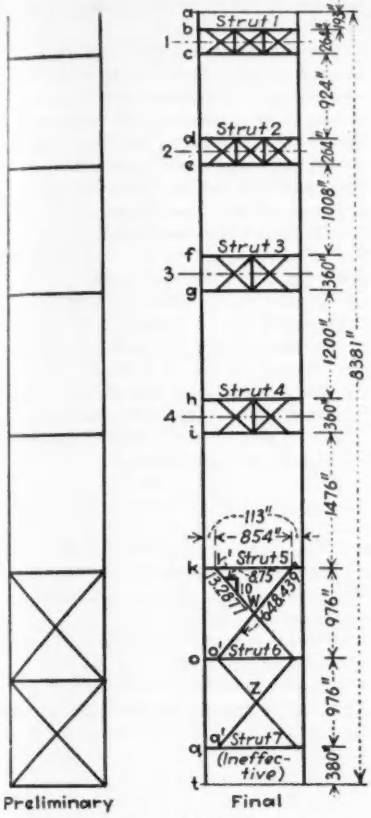


FIG. 4—TOWER DIAGRAMS used for preliminary and final calculations.

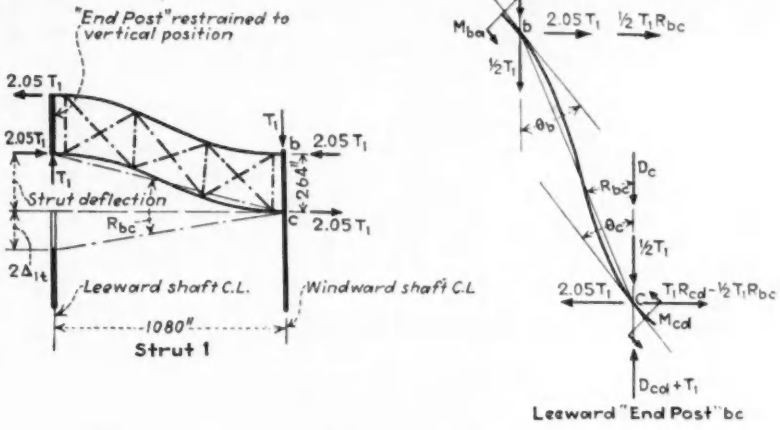


FIG. 5—SHAFT DEFLECTION ANGLE between chords of struts was expressed in terms of the strut deflection from a vertical upward load equal to the shear T_1 and the relative change in length of the two shafts, $2\Delta_{1t}$. The sketches portray the movements of the members involved.

tion angles of the two shaft sections, the two effective horizontal struts and the four diagonals in the fully braced section below the floor. The development of these equations is the most interesting feature of the study.

First Williot diagrams were drawn for the movement with respect to the base of all points below the floor. One of these diagrams is shown in Fig. 6. From these Williot diagrams the deflection angles of these 8 members were expressed in terms of the strains in the members and the angles of rotation at points k, o and q.

A typical expression for the diagonals is Eq. 1 below. Eq. 2 gives a typical expression for the shaft members, while a typical expression for the horizotal struts is:

$$R_{oo} = -\frac{1}{540}\Delta_{ot}$$

This gives the set-up of the 33 equations finally used. However, equations for the stresses and strains in the members had to be developed and introduced in these equations before they could be solved.

The two horizontal struts at k and o were assumed to have no direct stress from wind. The typical expression for the stress in a diagonal is illustrated by Eq. 3.

The expressions for the stresses in the shaft members were first obtained in the form of the stress added at each point. A typical expression for a point below the floor is shown in Eq. 4.

Eq. 5 shows a typical expression for the load added to the shaft at a portal strut above the floor or the reaction from the strut (say strut 2).

The summation of these expressions for the load added at various points gave the total wind stress in the shaft members.

The strains were computed from these stresses and introduced into the 33 basic equations after the moments were expressed in terms of the slope deflection unknowns and the rotation and deflection angles of the various members. The 33 unknowns consisted of 30 angles and the strains in members ko, oq and qt

$$R'_{qs} = -\frac{13.2877}{8.75}\left[\frac{10}{13.2877}\Delta'_{qs} + \Delta_{qt} + 113\theta_q\right] \div \left[\frac{13.2877}{8.75} \times 427\right] = -(0.7525757\,\Delta'_{qs} + \Delta_{qt} + 113\theta_q) \div 427 \quad (1)$$

$$R_{oq} = -\left[\left(\frac{10}{8.75} \times \frac{10}{13.2877} + \frac{8.75}{13.2877}\right)(\Delta'_{os} + \Delta'_{qs}) + \frac{20}{8.75}\Delta_{ot} + \frac{10}{8.75}\Delta_{oq} + \frac{10}{8.75} \times 113\theta_o + \frac{10}{8.75} \times 113\theta_q\right] \div 976 \quad (2)$$

$$= [1.32877(\Delta'_{os} + \Delta'_{qs}) + 2\Delta_{ot} + \Delta_{oq} + 113\theta_o + 113\theta_q] \div 854$$

$$\frac{8.75}{13.2877}S'_{kw} = 2778000 - 77413000\,R_{ko} + \frac{M_{ko} + M_{ok}}{976} + \frac{10}{13.2877}\left(\frac{M'_{kw} + M_{wk}'}{648.439}\right) \quad (3)$$

$$\text{Stress added at } 0 = \frac{10}{13.2877}S'_{ow} + \frac{8.75}{13.2877}\left(\frac{M'_{ow} + M_{wo}'}{648.439}\right) + \frac{M_{oo}}{540} + \frac{10}{13.2877}S'_{os} + \frac{8.75}{13.2877}\left(\frac{M'_{os} + M_{so}'}{648.439}\right) \quad (4)$$

$$\text{Reaction from strut 2} = \frac{-M_{oo} - M_{of} + 1104000 \times 264 - 64365000 \times 264\,R_{oo}}{540} \quad (5)$$

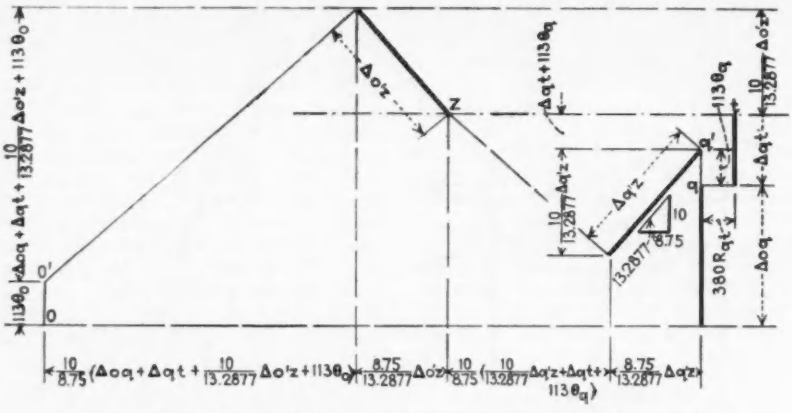


FIG. 6—WILLIOT DIAGRAM for bottom panel of bracing, typical of those used to obtain the deflection angle equations for the eight members below the floor.

which were kept as part of the final equations since they entered into several of the equations.

This method of analysis was so satisfactory that the time spent in earlier approximations was regretted. In any future design of this type this method of analysis would be used immediately after the first rough approximation by assuming a point of contraflexure midway between struts. The transverse wind loads and moments on each shaft as designed are shown in Fig. 8.

Other stresses

Following the determination of the sections from dead and live loads, wind loads and longitudinal bending stresses, the towers were investigated for other stresses. In addition to the effect of the changes in temperature on the distribution of the suspended loads between the cables and the stiffening trusses and on the longitudinal position of the tower tops, consideration was given to the effect of a difference in expansion or contraction between the pier and the tower corresponding to a difference of 40 deg. F. in temperature. The method of analysis was similar to that for the determination of stresses from transverse wind.

As a result of these studies it was decided that strut 7 should be made ineffective for carrying stress. The final stresses in the diagonals in the lower panel amounted to 1,300,000 lb. The maximum bending moment in the tower shaft was found to be 750,000,000 in.lb. at the base of the tower.

The effect of the participation of the bracing below the floor in carrying the direct dead and live loads on the shafts was determined in a manner similar to that for the determination of temperature stresses, and the equations were nearly identical. The participation stresses in the diagonals averaged about 3,500,000 lb. The maximum bending moment in the tower shaft amounted to 725,000,000 in.lb.

A study was made to determine the

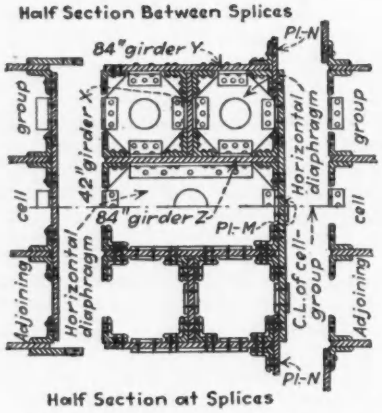


FIG. 7—MAKE-UP of typical cell group for one of the shafts of the Golden Gate Bridge towers. Cells are 3½ ft. square. Plates are ⅞ in. thick generally and angles are 8x8x⅞ in.

effect of unequal longitudinal deflection resulting from live load on one half the roadway. The primary purpose of this study was to determine the effect of such a condition on the lacing of the chords of the struts. The maximum difference in the deflection of the two shafts was found to be 8 in. Since there was uncertainty as to the torsional resistance of the cellular shafts stiffened by horizontal diaphragms, it was decided that the study should be made on the assumption that the shaft was as effective as a circular section of equal polar moment of inertia. Subsequent model studies indicated that the cellular shafts were about half as effective as was thus assumed.

It was found that twelve simultaneous equations could be written, the solution of which would give the loads on the shaft from the web members and the cable directly.

The longitudinal tower deflections at each of the twelve panel points for an unknown load at each panel point were determined. From these deflections were written simple expressions in terms of the unknown loads, for the deflection angles of the web members acting as horizontal beams.

The angles through which the shafts twisted at each panel point were found for a couple applied at each point. The couple at each panel point was equal to half the tower width times the unknown load except at the top where the couple was equal to the restraint of the cable or the horizontal component of the cable stress times the horizontal length of the effective part of the saddle multiplied by the unknown angle of twist. By equating this unknown angle to its value computed by applying the couples to the shaft, it was possible to eliminate this unknown angle of twist leaving only the twelve loads as unknowns. These torsional angles are also the angles of rotations of the horizontal beams comprising the web members.

From the expression for the deflection angles and rotation angles were written the expressions for the shears or end reactions of the horizontal beams which were equated to the unknown loads giving 11 of the 12 necessary simultaneous equations. The twelfth equation was found by equating the desired deflection at the top of the shaft less the deflection due to change in cable stress to the expression previously found for the tower deflection from loads at the panel points.

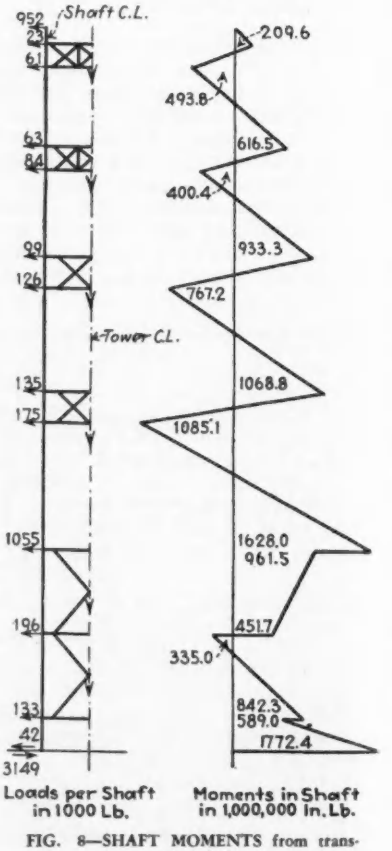


FIG. 8—SHAFT MOMENTS from transverse wind.

A solution of these twelve simultaneous equations gave directly the shears in the struts due to the differences in shaft deflections.

Although the process of determining the stresses appears somewhat elaborate for the determination of stresses that were realized to be subject to a considerable but probably fairly constant percentage of error, the investigation seemed justified since it revealed, quite accurately, the maximum possible stresses and the distribution of the stresses.

As a result of the investigation the three top struts were laced together in pairs reducing the resistance to torsion greatly. The lacing of the web members was increased as indicated by the results of these calculations.

It was realized that the secondary stresses in the members of the portal struts above the floor (Nos. 1, 2, 3 and 4) were relatively high and that the resistance of the individual members to bending would contribute appreciably to the stiffness of the struts; that changes in length of the shaft between the chords of a strut would produce stresses in the strut members; and that there were other small stresses due to the change in slope of the deflected shafts in the distance between the chords of the struts.

It was desirable to keep the same rigidity of the struts that had been assumed when the transverse wind stresses were computed so as to avoid a recalculation of these stresses. At the same time, it was not wanted to waste material or permit excessive unit stresses in any member. Previous approximate calculations had indicated the probable amount of these stresses, and the sections in general were thought to be satisfactory.

Starting with the assumed sections, the secondary stresses were computed by moment distribution in the conventional manner. The stresses due to changes in length of the shaft were found by the ratio of the shaft strains to the deflection of the strut from equal and opposite vertical loads applied to the strut at the tower connections. The effect of the angular changes in the shaft was computed. The total unit stresses were computed. The summation of unit stresses indicated a few desirable changes in section and in the number of panels in the struts, and the calculations were revised to take care of these changes.

Following this part of the work which resulted in sections stressed under these assumptions to a reasonably uniform unit stress, the effect of the shears from secondary moments on the direct stresses was determined. This study indicated that the struts were slightly stiffer than had been assumed when making the calculations for transverse wind and the sections were slightly decreased to keep close to the original assumption for the rigidity of the struts. All the previous strut calculations were then revised for the new sections and the procedure continued until the assumed sections gave the proper rigidity.

The effect of the gusset plates on the stiffness of the members of the struts was included in the determination of the moments of inertia and areas of the various members. Since their effect was appreciable, it was realized that the rigidity of the struts as a whole could not be determined exactly. The procedure adopted gave accurate results within the limitations of the assumptions made. Previous studies indicated that a small variation in the stiffness of the struts had little influence on the stresses in the structure as a whole.

Earthquake considerations

While the determination of stresses due to dead load, live load, wind and temperature presented a well defined problem subject to entirely satisfactory solution, it must be admitted that stresses due to seismic forces could not be evaluated with equal certainty. The towers, homogeneous, flexible shafts of steel, are anchored to massive concrete piers which are founded on rock. Although no one can predict just how a flexible shaft of this character will respond to an earthquake, some conclusions can be drawn as to its stability under these forces.

In the judgment of engineers who have investigated destructive effects of earthquakes, these towers may be subjected to earthquake vibrations of very small amplitude (a fraction of an inch) and accelerating forces amounting possibly to 5 per cent of gravity. In the completed structure the transverse deflection of the towers under the design wind load is more than ten times any expected movement of the pier tops, and the stresses from transverse wind will be more than double the stresses due to transverse earthquake forces. Due to the great flexibility of the towers in the longitudinal direction, stresses from longitudinal earthquake forces (5 per cent gravity) will not exceed 50 per cent of the longitudinal wind stresses.

The influence of earthquake forces on the design of the towers was a determining factor only in proportioning the anchorage to the piers so as to provide against earthquake while the towers were standing alone on the piers without the stabilizing effect of the cables and the suspended structure.

The gross area of each tower shaft at the base is 12,391 sq.in. and at the top 3,659 sq.in. As shown in Fig. 3, despite the fact that the four central transverse webs, to which the transverse bracing is directly connected, are made up of three widths of plates, they are, by splicing, made as effective as single plates having a width equal to the full width of the shaft. Filler plates ½ in. in thickness are used back of angles in locations where vertical joints occur in web plates. This makes it possible for tension plates to be inserted where it is desirable to carry tension across the joints without an offset or break in the main angles.

The tower shafts were fabricated in units which comprise "cell groups" of from three to six cells each in cross-section and generally about 40 ft. long. This grouping gave units within the working limits of major bridge shop facilities and also gave shipping pieces which were satisfactory in dimensions, weight and rigidity for handling. The largest units were about 7x10½ ft. in section, 45 ft. long and weighed 80 tons.

At horizontal splices the principal elements of the sections spliced, with few exceptions, are in direct bearing, and the splices are designed to develop 50 per cent of the compression strength in the sections spliced. An exception is that splices on extreme outer faces, both transverse and longitudinal, are designed to develop 60 per cent of the value of the section in compression.

Wherever the section of the tower shaft was reduced by omitting outer cells, a gradual transition from the larger to the smaller section was accomplished by lessening or tapering off, over a length of about 21 ft., the cross-sectional area of the cells omitted. This was considered necessary to avoid undesirable "hard spots" at these points and also to avoid overstress in the rivets at a point of abrupt change of cross-section.

Inasmuch as the transverse bracing connections are made directly to the four central transverse webs of the tower shafts, it is necessary to provide means for distributing to the other transverse webs their proportional part of the loads transferred at these connections. This was done through three planes of horizontal diaphragms placed in the tower shafts opposite each bracing connection. The diaphragms are tied together by means of tension plates so that collectively they form an effective horizontal girder which is designed to carry to each transverse web of the shaft its share of the transferred load.

Manholes provided in the cell webs, to give access to all cells, are about 70 ft. apart and are staggered for safety. Those in the exterior faces of the shafts are completely covered, while those in the interior webs are left open and are fully reinforced to make good the section cut away. At frequent intervals, openings 6½ in. in diameter were made in the web plates to facilitate delivery of field rivets to the interior cells.

Except for one cell in the center of each shaft, all cells are provided with horizontal diaphragms at intervals of from 12 to 15 ft. Each of these diaphragms is made up of one plate and four angles and is riveted to web plates on all four sides. The diaphragm plates were milled accurately to bear against the web plates so as to serve as spacers; the angles were set back ⅛ in. A ⅛-in. filler was used between the angles and the web plates wherever these were joined by field rivets, as it was found

that tight field rivets could not be obtained otherwise. These diaphragms were effective in obtaining accurate spacing of the webs and in maintaining the rectangular form of the cells.

Each tower is effectively anchored to the pier by riveted connections with pairs of angles imbedded for a depth of 52 ft. in the pier concrete. Also there are 78 dowels 6½ in. in diameter imbedded in the pier concrete and projecting through the 5-in. base slabs, to form a positive shear connection between tower base and pier. The angle anchorage was arranged so that it could be riveted directly to the longitudinal web plates of the tower shafts while an initial tension was maintained in the angles by means of jacks.

In the completed structure the cable saddles are permanently centered in both longitudinal and transverse direction on the tops of the tower shafts. During erection, however, they are set slightly to shoreward so that, as load is applied to the cables, the saddles can be moved channelward (with respect to the tower-tops) and thus brought to final position by progressive stages. To facilitate this movement a nest of rollers 8 in. in diameter was placed under each saddle. As originally set the saddles are shoreward, on the San Francisco tower 5 ft. 6 in., and on the Marin tower 3 ft. 7 in. Temporary structural supports for the saddles in their "off center" position are arranged so that their movement can be controlled by means of hydraulic jacks. When the cables are strung and loaded and the saddles take their final position they will be bolted permanently to the towers and the spaces between the rollers will be filled with some suitable solid material. The temporary saddle supports which are in the nature of projecting brackets will then be removed.

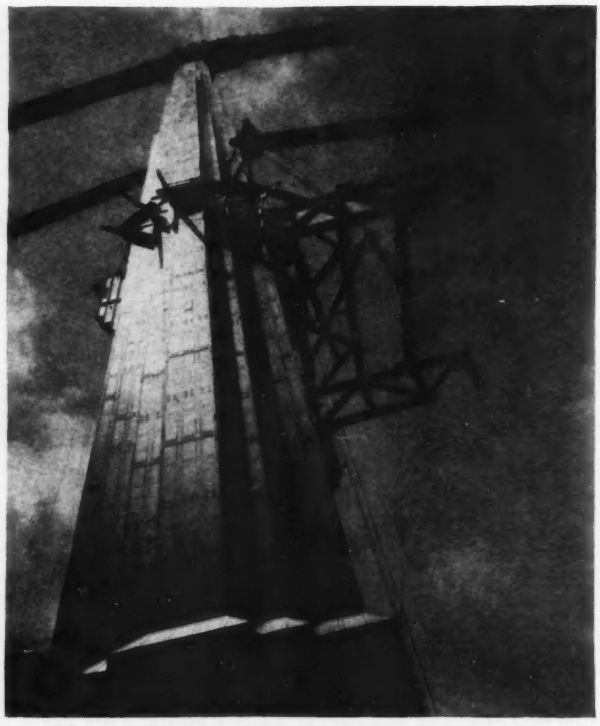


FIG. 9—STRUCTURAL MODERNISM as portrayed by the rivet patterns and functional setbacks on a transverse face of a shaft of one of the Golden Gate Bridge towers.

Fabrication of towers

The fabrication of the cell-group whose assembly is shown in Fig. 7 is typical for all cell groups. The sequence of fabrication operations was as follows:

1. Horizontal diaphragms were milled, assembled and riveted. Allowance was made in the milling of diaphragm plates to compensate for any variations in thickness of the main web plates so that in the complete assembly the correct dimensions between boundary planes of the piece would be obtained.
2. Girders X, 42 in. deep, were fabricated.
3. Girders Y and Z, 84 in. deep, were fabricated.
4. Girders X, Y and Z were assembled in a frame with diaphragms and plates M and N.
5. Horizontal diaphragms were reamed and driven.
6. Countersunk tack-rivets in plates M and N were driven, the holes having been subdrilled to templet.
7. The ends of the section were milled.
8. Field holes except splices were reamed or drilled from the solid.
9. The member was assembled end to end with the adjacent members of the same cell-group above and below. Splice material was fitted and reamed or drilled from the solid and then riveted at the upper end of the section.
10. After being separated from the adjacent piece above, the member was painted and shipped.

Horizontal splices of cell-groups are staggered with those of adjoining cell-groups to which they are joined by vertical seams of field rivets. All rivet spaces parallel to the column axis were made a multiple of a basic space of 1¾ in. and a metal "master templet" was made, as long as the longest shipping section and as wide as the widest, which contained all possible holes. This master templet was made with great care, all edges milled and holes fitted with hard steel bushings.

From the master templet all working templets were made. These had planed ends and were carefully measured and scribed to give exact lengths and width of section to which they were applicable. These templets were applied to single members for reaming or drilling from the solid all field holes except the horizontal splices at the ends of the members (operation 8 in the foregoing list). The latter were reamed through a "jumper templet," which was made to include two rows of seam holes immediately above and below the splice (operation 9 in the foregoing list).

As early as possible after fabrication by this means, about 60 per cent of the lower tier cell-group sections of one tower leg were erected at the shop and the assembly carefully checked to determine whether the desired results were being obtained. This trial erection demonstrated the sufficiency of the method and subsequently, in the field, the work fitted together in a most satisfactory manner.

Erection

Fabricated steel was shipped from the McClintic-Marshall Corp.'s plants at Steelton, Pa., and Pottstown, Pa., by rail to Philadelphia and thence by water, via the Panama Canal, to San Francisco. It was unloaded and stored at Alameda. Steel was delivered to the erecter as required in barge loads of 400 or 500 tons, which was the limit of storage at the tower piers. An 85-ton stiffleg derrick, with a 100-ft. boom alongside the tower, unloaded the barges and rehandled the steel as necessary to serve the erection traveler.

Preparation of the pier top to receive the tower base slabs was a part of the work included in the tower contract. The specifications required the top surface of the concrete under the tower base to be ground to a level plane. This was done with a carborundum grinding wheel mounted on a trolley. The trolley-supporting bridge itself was arranged to move on rails accurately leveled. With this equipment the bearing area under each tower shaft, amounting to about 1,200 sq.ft. was finished to the required level plane with variations of less than 1/32 in. To the finished pier surface a layer of red lead paste was applied immediately before setting the base slabs of which there were 19 per shaft.

The erection traveler comprised two steel trusses, 79 ft. long and 33 ft. deep, extending transverse to the bridge, spaced 28 ft. apart, and joined together with suitable cross bracing and struts. The traveler supported two stiffleg derricks, each of 85-ton capacity, equipped with 90-ft. booms. Provision was made in the tower shafts to support the traveler at predetermined elevations about 35 ft. apart. Preparatory to raising the traveler from one position to the next, two beams were placed across the uppermost tower sections, one above

each end of the traveler. Each end of these beams was equipped with four sheaves and these, with four corresponding sheaves at the lower four corners of the traveler, were reaved with 1-in. wire rope leading to the hoisting engines on the pier.

The derricks were independently operated by two electric motor-driven, four-drum hoists, located at the base of the tower. Each derrick had its own raising gang and electric light signal system for transmitting hoisting signals from the derrick to the engineer below.

Riveting of the horizontal splices in the tower shafts proceeded tier by tier. As soon as all horizontal joints of a tier of column sections were brought to proper bearing with the tier below, riveting of these splices was commenced. When the opening between milled ends of adjoining sections was reduced to 0.006 in. or less, the joint was considered to be in proper bearing. Where joints could not be closed within the above limit, but were open not more than 0.015 in., they were heated and riveted. These were few in number. There were a few instances where special means had to be adopted in order to close the joints as required. Usually this was effected by applying heated, structural hitches to the two sections so that they would be drawn together by the hitches as they cooled. Rivets in the vertical seams were not driven until rivets in all horizontal joints below them were driven.

There were about 600,000 field rivets in each tower. Rivets were heated by coal forges located on scaffolds outside the tower and were passed to crews within the tower by means of pneumatic tubes connected to the compressed air supply at the forge and extended through holes 6½ in. in diameter left in the cell walls for that purpose. As a protection against lead fumes arising from contact of hot rivets with red lead paint, all riveting gangs in the interior cells were required to wear respirators. All workmen wore hard fiber hats. Every possible precaution was taken to minimize hazards of erection. It is believed that the erection of these two towers, totaling some 44,000 tons of steel, without a fatal accident, establishes a noteworthy record.

GOLDEN GATE BRIDGE ARTICLES PREVIOUSLY PUBLISHED IN *ENR*

Design	May 28, 1931, p. 890
Construction	April 26, 1934, p. 541
Foundations	Aug. 22, 1935, p. 245
Cables	April 30, 1936, p. 617

The construction of the towers was part of the work to be performed by the McClintic-Marshall Corp. under its contract to furnish and erect all of the structural steel work for the main bridge at a contract price of $10,494,000.

Owing to the magnitude of the towers and the unusual character of the design, it was thought desirable to have a model built for the purpose of checking its general behavior under the different conditions of load to which it would be subjected. The model was built of stainless steel on a scale of 1 to 56. Proportionate loads were applied to the model and unit stresses were measured by strain gages. The results obtained from the model checked closely with the analytical results. The model was built by the Budd Manufacturing Co. of Philadelphia, and the model tests were made at Princeton by Prof. George E. Beggs (*ENR*, Jan. 25, 1934, p. 125).

Personnel

All work is carried out under the direction of Joseph B. Strauss, chief engineer. The writer is in direct charge of design and construction. From the engineering staff of Strauss & Paine, Inc., key men assigned to the work are: Russell G. Cone, resident engineer, and Charles Clarahan, Jr., and Dwight N. Wetherell, assistant engineers. Mr. Clarahan deserves most of the credit for discovering and applying the Williot diagram in the development of the strain equations.

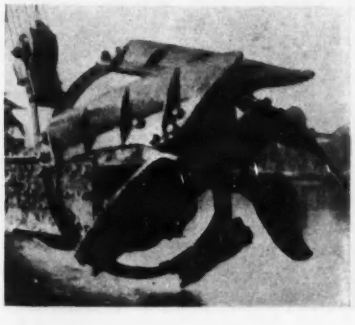
Collaborating with the chief engineer is an engineering board composed of O. H. Ammann, Charles Derleth, Jr., and Leon S. Moisseiff. Irving F. Morrow is consulting architect.

Dredge Cutter Head Handles Rock

A SUCTION DREDGE cutter head, with blades 6 ft. long designed for operation in sand and clay, recently was put to a severe test in dredging a ship channel near Willapa, Wash., when a considerable quantity of sedimentary rock in place was broken out and removed by the dredge. Although repairs and maintenance on the equipment were increased some $5,000 by the extra wear and tear incidental to handling rock, this was considered less expensive than delaying dredging operations and blasting out the rock.

The work was undertaken for the U. S. Engineer Department by contract and involved the removal of about 1,069,600 cu.yd. of material incidental to straightening the channel in Willapa Narrows. The bid on the excavation was 17¼ cents, a flat figure for the total yardage, of which 75 per cent was stiff clay; most of the remainder was closely packed sand. The channel was dredged to a width of 300 ft. and to a depth of 24 ft. below mean lower low water. Specifications allowed a 3-ft. overdepth in rock as against 1-ft. overdepth in soft material. In the softer material there was an area in which sedimentary rock projected some 6 ft. above the bottom of the proposed channel. This formation was classified as solid rock. After being passed through the dredge it was still solid enough to be crushed and used for road metal.

CUTTER BLADES after handling 2,000 cu.yd. of rock



At the outset of the work the intent was to uncover the rock with the dredge and then remove the dredging equipment while other floating equipment was brought in to drill and blast the ledge. However, as the job progressed and dredging was done over the ledge, the dredge began to bring up fragments of the rock with the softer material. The rock percentage increased without serious consequences and finally it was decided that because of the presence of the overburden mixed in with the rock, damage to the equipment probably would not be of a destructive nature. Operation of the cutter head therefore was continued cautiously until the entire rock ledge, an estimated total of 3,000 cu.yd., had been removed. Blades were damaged repeatedly; on one occasion one of the blades was broken out and never recovered.

The cutter head used in this operation consisted of six straight blades each 6 ft. long designed for operation at 23 r.p.m. and driven through a gear train by a 400-hp. alternating current motor. Ordinarily this cutter, operating four 6-hour shifts, removed 10,000 cu.yd. per day of soft material with the aid of a 1,500-hp. pump delivering through a 20-in. discharge line. The discharge was sometimes as long as 6,500 ft. although during work in the rock section a discharge line only 2,000 ft. long was used.

The contract was held by the Puget Sound Bridge & Dredge Co., of Seattle, and was carried out for the U. S. Engineer Department under the direction of Lt.-Col. Herbert J. Wild, district engineer, Seattle, Wash.

Domestic Wastes Treated In Industrial Sewage Plant

Compact layout of Imhoff tank, sludge drying bed and slow sand filters characterizes plant built for an isolated industrial community of 2,000 workers at the new works of the Electro Motive Corp.

PLACING an industrial plant in a rural setting, with room for expansion and small village living conditions for the workers, has its advantages from a manufacturing standpoint but it introduces special problems not common to urban locations. Among these problems which confronted the Electro Motive Corp., whose new plant near La-Grange, Ill. (*ENR*, July 16, 1936, p. 92), fits the above description, was that of sewage disposal. The solution adopted involves gravity flow of the domestic wastes to a sump or wet well, a pumped influent line 1,400 ft. long to a layout consisting of an Imhoff tank, sludge drying bed and slow sand filters and an effluent discharge into a nearby storm sewer.

The plant, planned for a daily flow of 30,000 gal. from an ultimate worker population of 2,000, was designed according to the most modern thinking available. Built at a time when the Illinoise authorities were planning sanitary facilities with work relief funds for many communities, the owners of the plant cooperated with the state department of health to the end that the new plant should serve as a standard for small sewage disposal projects. It has in fact served in this capacity on several occasions. The layout is shown in the accompanying plan.

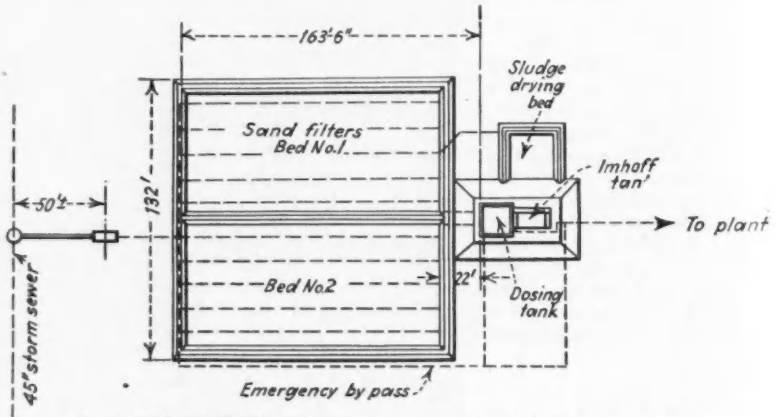
The collection system

Cast iron soil pipes and vitrified clay mains comprise the collection system. The mains, installed in trenches beneath the concrete floors of the building, are laid on grades that provide self-cleaning velocities to a wet well or sump in the base of the boiler house smoke stack. This novel location conserved space and at the same time permitted economical construction since only a base slab had to be built, the walls being provided by the stack. The sump has a maximum capacity of 675 gal. and a working capacity of about 400 gallons.

In an adjacent room two 3-in. heavy duty vertical sewage pumps with automatic controls are installed. Each pump has a capacity of 125 g.p.m. against a 40-ft. head, and discharges into a 6-in. cast iron main which carries the sewage 1,400 ft. to the disposal plant at the edge of the site. The total static and friction head of the line is about 30 ft.

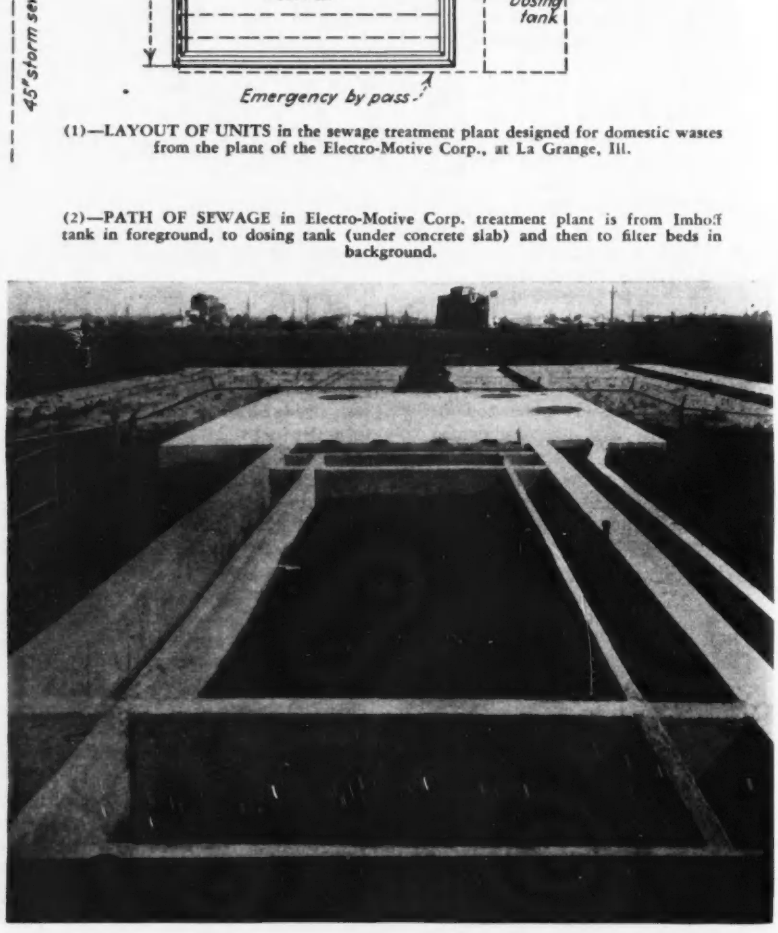
At the plant the sewage enters an Imhoff tank whose sedimentation chamber is designed for a 3-hr. detention period based on the average hourly flow. Provision is made for reversal of flow through the chamber. The sludge digestion chamber has a capacity of about 1,800 cu.ft. or 3 cu.ft. per person on the basis of the initial plant population. This unit capacity will be reduced considerably when the ultimate manufacturing facilities are developed, and it is probable that an additional Imhoff tank will then have to be provided. Digested sludge is withdrawn by hydrostatic pressure and means have been provided for breaking up and softening the sludge whenever it becomes hard at the bottom.

From the sedimentation chamber the sewage flows to a dosing tank with capacity below the discharge line of 9,100 gallons, an equivalent of 7.1 hr. average flow and covers each filter bed once every 14.2 hr. to a depth of 1.138 ft. at the average rate of flow. This dosing chamber is equipped with two siphons of the double alternating type



(1)—LAYOUT OF UNITS in the sewage treatment plant designed for domestic wastes from the plant of the Electro-Motive Corp., at La Grange, Ill.

(2)—PATH OF SEWAGE in Electro-Motive Corp. treatment plant is from Imhoff tank in foreground, to dosing tank (under concrete slab) and then to filter beds in background.



and with a heavy-duty float-operated discharge counter. This counter has a 4-in. copper ball carrying a ¼-in. brass rod enclosed in a 5-in. pipe and is covered with a waterproof wood housing.

Filters and sludge drying bed

The filters which receive the sewage from the dosing tank consist of two sand beds, each 66 ft. x 139 ft. Excavation for the filter beds was made exactly according to sections as shown on the drawings, and the slopes were finished carefully by hand, good soil being tamped in at all soft and spongy spots until a solid surface had been formed. The drain trenches likewise were accurately cut to line and grade.

The sand beds as constructed consisted of 24 in. of clean, washed, quartz sand having an effective size not less than 25 mm., underlaid with 6 in. of crushed stone beneath which 6-in. underdrains are laid in the bottom of the valleys 12 in. deep which are filled with crushed stone of 1 to 2½-in. size. The underdrains lead to a main collector across one end of the filters, this collector discharging into a 54-in. storm sewer across the north end of the 70-acre plant site. In operation of the plant each filter bed is covered once every 14.2 hr. to a depth of 1.138 ft. assuming the average hourly rate of flow of sewage to the plant.

The sludge drying bed, 25 x 25 ft. in plan, consists of a 12-in. sand layer on top of 6-in. tile drains laid in crushed stone. On the basis of present plant population the sludge bed provides 1 sq.ft. per capita.

Two methods of bypassing have been provided for emergency use. When it is necessary to cut the filters out of use, a bypass from the dosing tank to the main collector is available through an 8-in. vitrified line. When it is necessary to bypass the entire plant, a 4-in. cast iron line is available, to connect the force main from the wet well to the collector. All gate valves are of cast iron body with brass line gate and non-rising stem designed for a working pressure of 40 pounds. All valves are provided with valve boxes and extension stems with operating handles.

Computing Volumes In Intersecting Arches

Problem of computing the volumes in intersecting arches solved by determining the common volume of the two segments of the cylinders intersecting at right angles

By Louis J. Sack
Assistant Engineer, Board of Water Supply, New York City

THE PROBLEM of determining the volume of masonry in two arches intersecting at right angles is one of the minor aspects of computing quantities, yet from the writer's observation it is one in which the engineer often finds considerable difficulty. Almost anyone who has had to do with estimating quantities for payment in work of this kind and, particularly, in subway construction, will recall the various methods used and the uncertainties involved. Actually the problem resolves itself into determining the common volume of two segments of circular or parabolic cylinders intersecting at right angles.

Fig. 1 represents the intersection of two parabolic segments referred to three rectangular co-ordinate axes OX, OY and OZ. Let S denote the span and r the rise of the two segments. The equations of the two segments are respectively

$$Y = \frac{4r}{S^2}X^2 \text{ and } Z = \frac{4r}{S^2}X^2$$

The element of the common volume formed by the intersection is represented by

$$dv = (r - Y)(r - 3)\,dx = \left(r - \frac{4r}{S^2}X^2\right)^2 dx =$$

$$r^2\left(1 - \frac{8}{S^2}X^2 + \frac{16}{S^4}X^4\right)dx$$

Total volume is obtained by integrating the above expression between limits $S/_2$ and $-S/_2$ as follows:

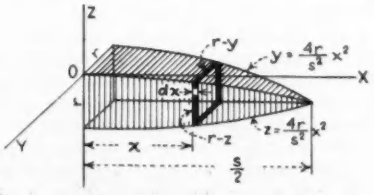


FIG. 1—INTERSECTION of two parabolic segments—S represents the span and r the rise of the arch.

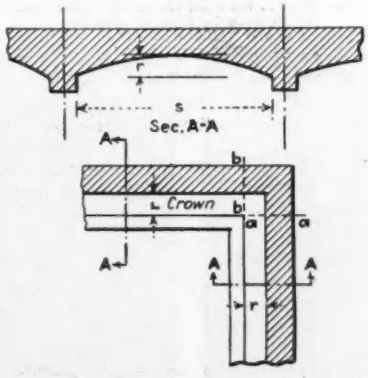


FIG. 2—SECTION OF JACK arch, of the type often used in subway construction.

$$V = r^2\int_{-S/2}^{S/2}\left(1 - \frac{8}{S^2}X^2 + \frac{16}{S^4}X^4\right)dx$$

$$= r^2\left(X - \frac{8}{3S^2}X^3 + \frac{16}{5S^4}X^5\right)\Bigg|_{-S/2}^{S/2}$$

$$= r^2\left(S - 2/3\,S + \frac{1}{5}S\right) = \frac{8}{15}r^2S \quad (1)$$

For segments of circular cylinders intersecting at right angles, the determination of the common volume is obtained in a similar manner, although the final expression for the volume is not so simple. Let r and S be defined as above and let R denote the radius of the circular arch. The radius expressed as a function of the span and the rise is represented by

$$R = \frac{r}{2} + \frac{S^2}{8r}$$

The equations of the two segments referred to axes as above are given by

$$X^2 + Y^2 - 2RY = 0 \text{ or } Y = R - \sqrt{R^2 - X^2}$$

$$X^2 + 3^2 - 2R3 = 0 \text{ or } 3 = R - \sqrt{R^2 - X^2}$$

The element of volume is

$$dv = (r - Y)(r - 3)\,dx = (r - R + \sqrt{R^2 - X^2})^2\,dx = [r^2 + 2R^2 - X^2 - 2rR - 2(R - r)\sqrt{R^2 - X^2}]dx$$

Total volume $V =$

$$= \int_{-S/2}^{S/2}[r^2 + 2R^2 - X^2 - 2rR - 2(R - r)\sqrt{R^2 - X^2}]\,dx =$$

$$\left[r^2X + 2R^2X - \frac{X^3}{3} - 2rRX - (R - r)\,X\sqrt{R^2 - X^2} - R^2(R - r)\,\text{Sin}^{-1}\frac{X}{R}\right]\Bigg|_{-S/2}^{S/2} =$$

$$r^2S + 2R^2S - \frac{S^3}{12} - 2rRS - (R - r)\,S\sqrt{R^2 - \tfrac{1}{4}S^2} - 2R^2(R - r)\,\text{Sin}^{-1}\frac{S}{2R},$$

But $(R-r)\,S\sqrt{R^2-\frac{1}{4}S^2} = S(R-r)^2$

Hence above expression reduces to

$$V = R^2S - \frac{S^3}{12} - 2R^2(R-r)\,\mathrm{Sin}^{-1}\frac{S}{2R} \quad (2)$$

A comparison of the common volumes of intersecting circular and parabolic arches of various ratios of rise to span is shown in the accompanying tabulation. In order to obtain the volume, these numerical coefficients are to be multiplied by the cube of the span (S^3). This tabulation shows that for a ratio of rise to span of 10 per cent, the variation in volume is 1.3 per cent, and for a ratio of 15 per cent it is 2.6 per cent. Since most arches in practice fall within these limits, the method of obtaining the common volume by the comparatively simple formula for parabolic segments is sufficiently accurate. If greater accuracy is desired, a correction is to be added depending upon the ratio of rise to span.

The following illustration, as shown in Fig. 2, will indicate the method of applying the formula to the computation of volumes. The figure represents a type of arch known in subway construction as jack arch. The computation is to be done as follows:

1. Compute the volume in the side wall arch to the line *a-a* (spring line of roof arch)
2. Compute the volume in the roof arch to the line *b-b* (spring line of side wall arch)
3. Compute volume between *a-a* and *b-b* as a rectangular solid
4. Deduct the common volume as obtained from the formula or tabulation.

COMMON VOLUMES OF INTERSECTING ARCHES

Multiply coefficients by cube of span (S^3)

	Ratio of rise to span (per cent)			
	10	15	20	25
Circular Arch...	0.00540	0.01231	0.02228	0.03562
Parabolic Arch..	0.00533	0.01200	0.02133	0.03333
Variation in volume (per cent)	1.31	2.58	4.45	6.87

Roof Repairs with Cotton Sheeting

Unbleached muslin fabric, properly applied with suitable paint, makes a tight, waterproof and flexible patch that gives years of satisfactory service

By Louis Deck, Jr.
Woodside, L. I., N. Y.

UNBLEACHED COTTON sheeting or muslin applied with paint has long been used for patching leaky roofs, but usually in a haphazard manner without proper materials. Though such repairs were slipshod, satisfactory results were obtained in small patches. Recently the use of muslin saturated during application has extended to repair of large roof areas, to skylights and to parapet walls and copings. It is proving ideal for this purpose because it is possible to set the fabric to withstand shock, vibration, and expansion and contraction of the supporting structure. However, on the larger installations and on skylights, haphazard methods of application will result only in poor and unsatisfactory work.

Briefly, satisfactory cotton sheeting roof repairs depend upon the proper grade and weight of muslin, the use of a suitable paint free of elements destructive to cotton fibre, and the correct application of paint and sheeting to the roof. Failure to observe these three important points is certain to result in a disappointing repair job.

Materials

The muslin should be unbleached, having a weight of 3½ to 4½ lin.yd. per lb., with a thread weave of 40 to 60 per in. in each direction. Such material is both flexible and tough. Flexibility is necessary to resist contraction and vibration stresses and to follow the roof contours, while toughness assures strength to bridge the gaps of holes and leaks in the supporting medium. This size weave has been found to be large enough to permit thorough saturation by the paint, and yet small enough to eliminate excessive voids in the fabric. Properly saturated and applied, muslin of these characteristics results in a flexible, yet waterproof and airtight membrane.

The proper kind of paint is of great importance. A medium thick paint, either bituminous or oil, is best. It should be acid-resisting, and, most important of all, should be free of chemicals injurious to cotton fibre. Obviously, the paint should preserve and not destroy the fabric. Paints that require thinning agents or which must be heated before being applied are not desirable. The primary purposes of the paint are to supply the cementing agent for holding the fabric to the original surface, to fill the voids in the muslin to form a waterproof membrane, and to preserve and protect the sheeting from action of smoke and the elements.

Many paints excellent for other purposes are unsuitable for use in cotton

FULL COVERAGE of a slate roof and repairs to skylight with cotton membrane. The view at the right shows the application of sheeting to skylight mullions.

REPAIR of a slate roof and metal ridge roll with saturated cotton sheeting. The light spots are the new patches.

membrane roofing. One common fault is that the fibre absorbs the oils in certain paints, and thus leaves the membrane porous and brittle. Others do not have proper cementing qualities, and still others actually destroy the fibre by chemical action. Paints with long fibrous fillers should be avoided, for such fillers span the voids and threads of the fabric preventing the proper filling and saturation of the sheeting.

Making the patch

Just prior to application, the muslin should be soaked in clean water for at least 24 hr., and preferably for 48 hr. This removes the sizing on the threads, and causes preliminary shrinkage. Fabric left in water too long, more than four or five days, will begin to rot. The sheeting is applied wet, as experience has shown that the wet fibres absorb the paint better than do dry threads, and that the wet sheeting sticks better and works easier.

The first step is to dry sweep the surface to be repaired to remove all dust and dirt. Then a coat of paint is applied cold (heating destroys valuable oils) by thorough rubbing and spreading on the old surface. Application is by a heavy, long-handled brush. The wet muslin is placed immediately upon the wet paint. Right here is where many a roofing repair job is spoiled. The sheeting must not be stretched tight, but is laid fairly loose, with plenty of wrinkles placed in the proper locations to allow for future contraction and expansion. In laying fabric in corners, or in valleys, on corrugated iron, slate or tile, a slack or crease of ½ in. is desirable. The sheeting should follow the surface and contours of the roof material closely, and stretching over low spots or across corners is to be avoided. Creases or wrinkles must be left over cracks, holes, laps and seams in the original roofing.

As soon as the muslin has been placed it is given another thorough application of paint. The paint should be worked into the fabric, through the expansion ridges, which are finally plastered down flat, and care exercised to remove all entrapped air bubbles. Paint thoroughly rubbed in is preferable to excess paint spilled on the fabric.

The patch is now complete, except that to assure satisfactory results another coat of paint should be applied within 30 to 60 days after the original application to seal off any possible cracks or voids. Thereafter it is wise to repaint the membrane at the end of the second, fourth, seventh, tenth, and fourteenth years, and then every five years. Under this occasional maintenance the membrane seems to improve with age. The succeeding coats of paint work well down into the previous coats, maintaining the muslin as an integral part of the whole paint thickness. There is no peeling of paint properly applied to the membrane.

Wide application

Patches and coverings of this type are suitable for any kind of roofing, except of course the stone must first be removed from gravel roofs. They have been successfully applied to wooden, slate, tile, gypsum, concrete and iron roof surfaces of all shapes, to metal flashings, and glass skylights, and to stucco parapets. Joints both vertical and horizontal, in copings and cornices of all kinds, have been waterproofed by this method. The impregnated muslin not only produces a waterproof surface, but it also stops the corrosion of iron roofs and checks disintegration of concrete tile. Condensation on the underside of roofs is greatly reduced.

The sheeting can be placed with specially-prepared colored paints and with care can be laid to follow the contours of Spanish tile and other artistic shapes. The best method of color application is to use a white or slightly tinted coating for the setting of the muslin, and use the desired finish color paint on top of the sheeting. This work naturally requires the utmost care in selection of materials and mixing of paints.

Properly laid with right materials, cotton membrane roofing will give years of satisfactory service. It adds less than ½ lb. per sq.ft. load to the roof, and is slow-burning because of its close adherance to the original surface.

Merit System for Government Employees

ACTION FAVORING the merit system on every governmental front —local, state and federal—is being taken in unprecedented degree, according to the September issue of the *Civil Service Assembly News Letter*, which reports current developments in civil service.

Beginning October 1, two-thirds of the employees of the Federal Rural Electrification Administration will come under civil service by non-competitive examination. Future appointments to the Federal Home Loan Bank Board and its subsidiaries are to be made under the supervision of the Civil Service Commission. This includes the Federal Home Owners' Loan Corporation with over 15,000 employees.

In Arkansas the Democratic nominee for governor, who is the present attorney-general, has named an honorary commission of three members to survey the operation of the merit system throughout the country and to recommend a law. An executive and accountant heads the commission, whose members are serving without pay. A newspaper editor and an attorney are the other members.

Arkansas is one of at least nine states giving serious thought to civil service for state, county or city employees. In two of these states—Washington and California—voters will make the decision at the polls in November. The Washington bill would blanket under civil service all governmental units. California's proposed act would make civil service mandatory for cities, and would include port, park and school districts. Alabama, Connecticut, Indiana, Michigan, Minnesota, and Virginia are the other states in which proposals for civil service laws may be placed before legislatures at their next sessions.

Spring Floods Test Winooski Flood-Control System

Two completed flood-control dams on the Winooski River in Vermont, begun following 1927 flood, show possibilities of the projected seven dams in the flood of the past spring.

By Charles D. Curran
Assistant Engineer, U. S. Engineer Office, First New York District, New York City

THE FLOODS of last March that resulted in widespread destruction throughout much of New England and other eastern states put to its first test the flood-control system now under construction on the Winooski River in Vermont, a system that had its inception in the disastrous flood that swept the Winooski valley in November 1927. Though only two of the projected seven reservoirs had been completed at the time of the March 1936 flood, these two materially reduced the flood peak and showed clearly what can be done with additional flood storage reservoirs and further channel improvements. No serious damage was done in the March storm, which though of slightly less intensity than that of 1927, produced a greater total runoff.

In June, 1933, following the organization of the Civilian Conservation Corps, Robert E. Fechner, Director of Emergency Conservation Work, authorized the employment of C.C.C. personnel on the construction of part of the Winooski River Flood Control Project in Vermont. Supervision of the engineering aspects of the work was vested in the Chief of Engineers, U. S. Army, under whose direction the comprehensive project, described in House Document No. 785, 71st Congress, 3rd Session was developed. This originally involved the construction of seven storage reservoirs, to be operated in the combined interest of flood control and water power development, together with supplementary work of channel improvement for the benefit of the cities of Montpelier and Barre, Vt. However, the plan as modified for employment of the Civilian Conservation Corps considered only four of the above reservoirs of which only three finally reached the construction stage (see *ENR*, Nov. 9, 1933, p. 155 and Aug. 30, 1934, p. 266). Two of these, namely at Wrightsville and East Barre, were completed and in operation in 1935, and it is expected that the third on the Waterbury River will be completed in 1937. The two completed dams were built somewhat smaller than originally planned, their operation being restricted to flood detention entirely, without any storage being reserved for use in the interest of water power development. Extensive channel clearing and improvement work also was completed in 1935 and an old wooden dam which obstructed the passage of the Winooski River flood waters through Montpelier was replaced by a modern concrete

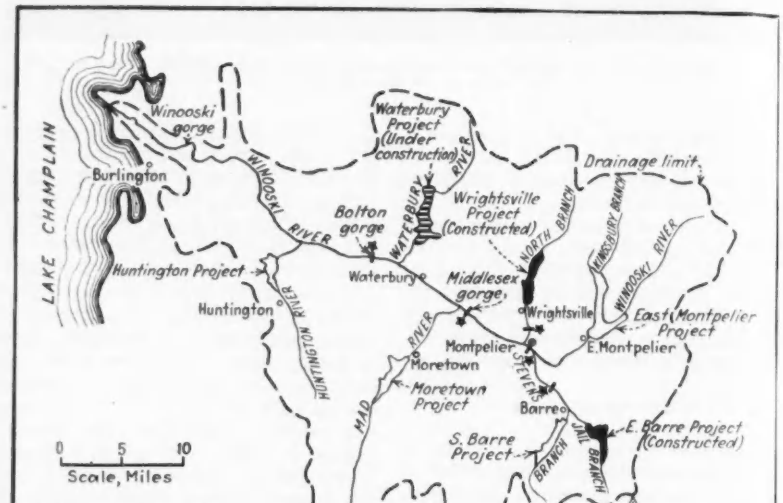


FIG. 1—WINOOSKI RIVER VALLEY showing the seven projected reservoirs, two of which were completed and in operation at the time of the March, 1936, flood.

FIG. 2—OUTLET CONDUIT at East Barre Dam discharging under conditions approaching the maximum. Satisfactory dissipation of the energy is being obtained by hydraulic jump.

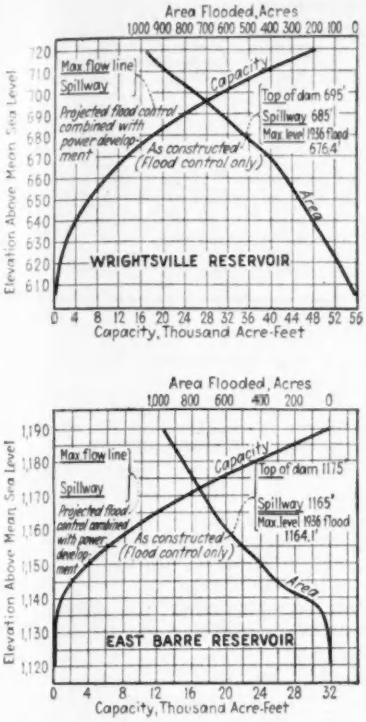

Area Flooded, Acres
1,000 900 800 700 600 500 400 300 200 100 0
Elevation Above Mean Sea Level
Max flow line
Spillway
Projected flood control combined with power development
As constructed (Flood control only)
Capacity
Area
Top of dam 695'
Spillway 685'
Max level 1936 flood 676.4'
WRIGHTSVILLE RESERVOIR
0 4 8 12 16 20 24 28 32 36 40 44 48 52 56
Capacity, Thousand Acre-Feet
Area Flooded, Acres
1,000 800 600 400 200 0
Elevation Above Mean Sea Level
Max flow line
Spillway
Projected flood control combined with power development
As constructed (Flood control only)
Capacity
Area
Top of dam 1175'
Spillway 1165'
Max level 1936 flood 1164.1'
EAST BARRE RESERVOIR
0 4 8 12 16 20 24 28 32
Capacity, Thousand Acre-Feet

FIG. 3—AREA-CAPACITY CURVES for the Wrightsville and East Barre reservoirs showing the elevation to which the 1936 flood went. Flow lines and spillway elevations as proposed for combined power and flood control are shown.

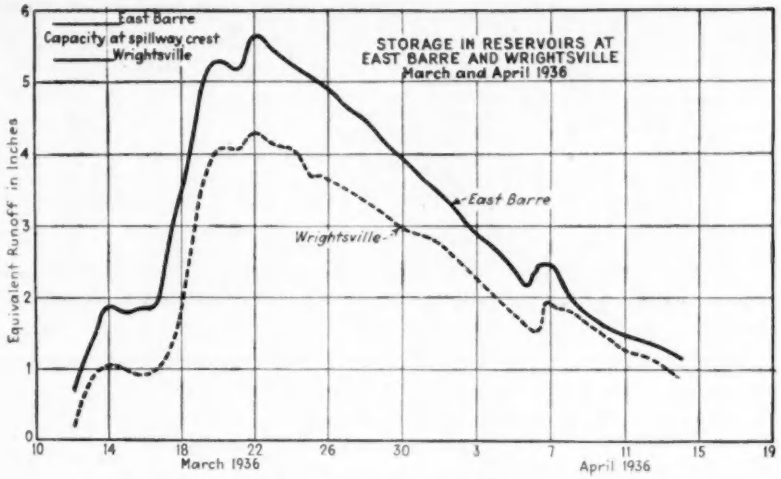

East Barre
Capacity at spillway crest
Wrightsville
STORAGE IN RESERVOIRS AT EAST BARRE AND WRIGHTSVILLE
March and April 1936
Equivalent Runoff in Inches
East Barre
Wrightsville
0 1 2 3 4 5 6
10 14 18 22 26 30 3 7 11 15 19
March 1936
April 1936

FIG. 4—STORAGE IN THE RESERVOIRS at Wrightsville and East Barre.

TABLE NO. 1—STORM OF MARCH, 1936

Date (March)	Above Wrightsville Dam Precipitation (Inches)	Above Wrightsville Dam Run-off (Inches)	Above East Barre Dam Precipitation (Inches)	Above East Barre Dam Run-off (Inches)
12	1.45	0.68	1.24	1.32
13	0.24	0.81	0.23	0.88
14	0.04	0.35	0.03	0.36
15	0.00	0.23	0.00	0.33
16	0.88	0.39	1.09	0.46
17	0.19	0.78	0.49	1.46
18	1.69	1.82	0.48	1.76
19	0.29	1.45	0.26	1.10
20	0.03	0.75	0.00	0.50
21	0.73	0.62	0.53	0.78
22	0.48	0.46	0.18	0.44
23	0.00	0.24	0.00	0.20
24	0.29	0.28	0.02	0.23
25	0.43	0.76	0.35	0.33
26	0.00	0.56	0.00	0.18
27	0.31	0.52	0.28	0.19
28	0.00	0.32	0.00	0.15
29	0.00	0.16	0.02	0.13
30	0.02	0.11	0.00	0.16
Total	7.04	11.30	5.20	10.96

structure with tainter gates to facilitate the passage of floods.

As constructed, the Wrightsville dam on the North Branch of the Winooski River north of Montpelier is a rolled-fill structure, with a drainage area of 71 sq.mi. Its crest is 1,200 ft. long. Maximum height is 70 ft., base width 700 ft., and top width 165 ft. It contains 860,000 cu.yd. of clay and selected material. Its capacity to spillway crest is 20,500 acre-ft. and its capacity to the maximum flow line, five feet below the top of the dam, is 23,000 acre-ft. With a 5-ft. surcharge, the 160-ft. spillway is capable of discharging 6,300 sec.-ft. It is estimated that a flood 40 per cent in excess of the one percent frequency flood would be required to give this surcharge.

The East Barre dam, with a drainage area of 39 sq.mi., is on the Jail Branch about 13 miles above Montpelier. Its height is 65 ft., bottom width 400 ft., and crest width 85 ft. It contains 275,000 cu.yd. of fill. The crest length is 1,600 ft. Capacity to spillway level is 12,000 acre-ft. and to the maximum flow line, five feet below the top of the dam, 16,200 acre-ft.

Both dams are provided with concrete conduits, open at all times, and of such capacity as to maintain the river within its banks while the reservoirs are filling during flood. Both dams have large crest widths to allow for future increase in height, should power development prove desirable. Area-capacity curves are shown in Fig. 3.

The dams were built to give partial control from floods similar to those which devastated the region in November, 1927. Their first test came less than six months after their completion with a flood which, while of lesser intensity, was of greater total runoff.

The winter of 1935-36 was relatively severe in Vermont. Precipitation in December and January was only slightly above normal but mean temperature was about 4 deg. below normal. On March 9 about 20 to 22 in. of snow covered the upper watershed above the two completed dams. For the week ending March 10, temperatures averaged 23 deg. F. and precipitation was about one-third of the normal. Beginning March 12, moderate rains, followed a week later by heavy rains and rising temperature, struck the eastern part of the United States. In the vicinity of the upper Winooski, the temperature rose to an average of 37 deg. for the week ending March 17 and to 41 deg. for the following week. This was 12 deg. above normal. During these same two weeks the precipitation, while not so excessive as that further east, amounted to about 4.7 in. at Northfield, Vt., or over four times the normal for the period. The sudden thaw, the great snow cover, the heavy precipitation and the frozen ground here, as well as in other valleys of New England, New York and Pennsylvania, caused a high run-off. Table I shows rainfall and run-off above the two dams, from March 12 to 30. Rainfall shown is an estimate based on records of nearby gages. In view of the uniformity of the rainfall during this storm, the error involved in the use of these estimates is not considered to be large.

On March 12 the ice started to break up at many points on the river and the first flood peak occurred. Early in the morning of March 17, between 1:00 and 3:00 a.m., the thick ice above the Montpelier dam broke up. The dam had been under careful observation of U. S. Engineer Department personnel, under an agreement with the state that the department would supervise the operation of the dam during the 1935-36 winter. Two of the gates had been kept free of ice by the use of compressed air and heating, so as to open within four hours notice. Four of the tainter gates

TABLE NO. II—PEAK FLOWS, WINOOSKI VALLEY, VT., STORM OF MARCH, 1936

Stream	Locality	Miles Above Mouth	Flood of March, 1936 Date	Time	Peak Discharge (Sec.-ft.) Total	Per Square Mile	Flood of Nov., 1927 (Sec.-ft.)
Winooski River	Molly's Pond Reservoir	84	18	1 pm	.....	.....	581
Steven's Brook	Above mouth of Jail Brook	65	18	4 pm	9,600	200	22,300
Jail Brook	Above East Barre Dam	68	18	12 pm	1,920	50.6	.....
Jail Brook	Below East Barre Dam	67.8	22	12 m	420	11	11,500
North Branch	Above Wrightsville Dam	61	18	12 pm	3,930	54.4	.....
North Branch	Below Wrightsville Dam	60.5	22	12 m	950	13.4	17,200
North Branch	Lane Mfg. Co. Dam	56.8	19	3 pm	1,200	15	19,200
Winooski River	Montpelier Dam	56	18	6 pm	.....	.....	50,400
Winooski River	Below Montpelier	55	18	8 pm	19,900	47.5	57,000
Dog River	Above Northfield	64	18	7 pm	4,350	90.6	8,000
Mad River	Below Moreton	55	18	4 pm	8,050	60.0	.....
Waterbury River	Below Dam Site	44	18	6 pm	5,000	44.6	.....
Winooski River	Below Essex Junction	15.3	19	12 m	47,000	44.7	111,000

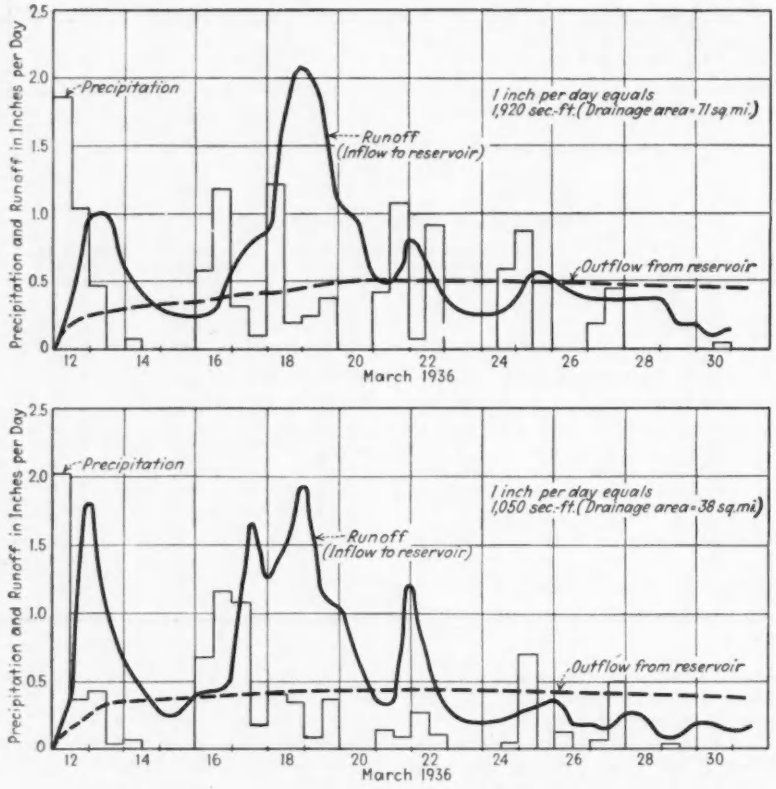
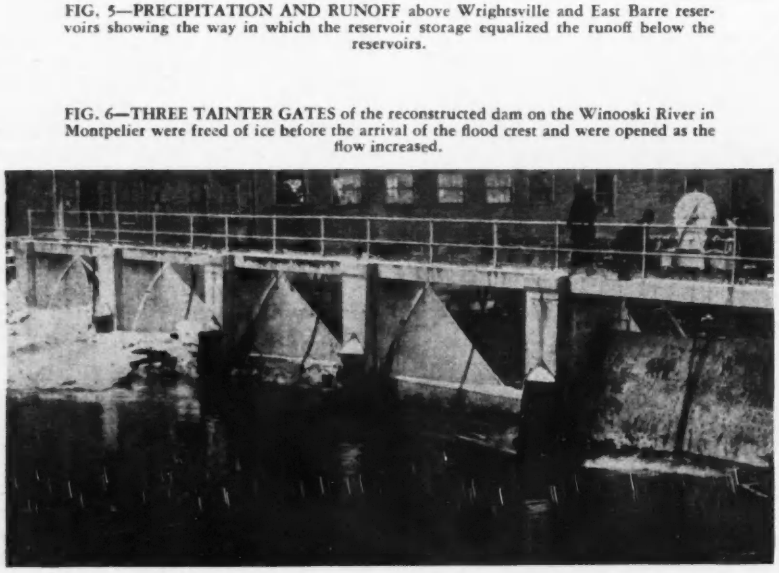


FIG. 5—PRECIPITATION AND RUNOFF above Wrightsville and East Barre reservoirs showing the way in which the reservoir storage equalized the runoff below the reservoirs.

FIG. 6—THREE TAINTER GATES of the reconstructed dam on the Winooski River in Montpelier were freed of ice before the arrival of the flood crest and were opened as the flow increased.



were promptly opened. Ice was dynamited as it lodged against the dam and was kept moving so that no serious trouble was experienced. It was, however, nearly 24 hours before the river and mill pond were cleared of ice. Although the ponds of power dams above had not yet filled to capacity, a great quantity of water passed over this dam, coming from the unregulated streams.

With the additional rising of the river, the remaining two tainter gates were opened, the last one on the morning of March 18. By noon of this day Molly's Pond Reservoir (29 miles above) had filled to its spillway crest but flashboards prevented discharge. The river at Montpelier rose to within 1¾ ft. of the tops of the tainter gates in their open position.

Late in the evening of March 18, the river height began to recede and by morning the level had dropped several feet. On the 20th flow was so reduced that one of the gates was closed. In the meantime, water was steadily rising behind the Wrightsville and East Barre dams. Fig. 4 illustrates the rate of storage in the two reservoirs. By March 22, the water behind the Wrightsville dam had risen to 64 ft. above the outlet conduit or to within about 8 ft. of the spillway crest, retaining about 16,000 acre-ft. or about 78 per cent of capacity. This represents 4.2 in. of run-off in the basin exclusive of about 3.8 in. that had been released through the outlet up to the time of maximum storage. The maximum outflow from the dam was 960 sec.-ft. which passed down the North Branch into the Winooski River. On Nov. 3, 1927, before construction of this dam, the flow at the same locality was 17,200 sec.-ft. Fig. 5 shows inflow-outflow hydrographs of this reservoir and rainfall over its drainage area.

At the same time water in the East Barre dam had risen to within 11 in. of the spillway crest, or to a height of 36 ft. above the outlet conduit, holding back about 11,200 acre-ft. or about 95 per cent of the capacity to spillway level. This amounts to 5.6 in. of run-off from the watershed above the dam, exclusive of 4.0 in. released through the outlet in the dam up to that time. The discharge on the Jail Branch below the East Barre dam was only 450 sec.ft. This compares with an unregulated peak in 1927 at the same locality of 11,500 sec.-ft.

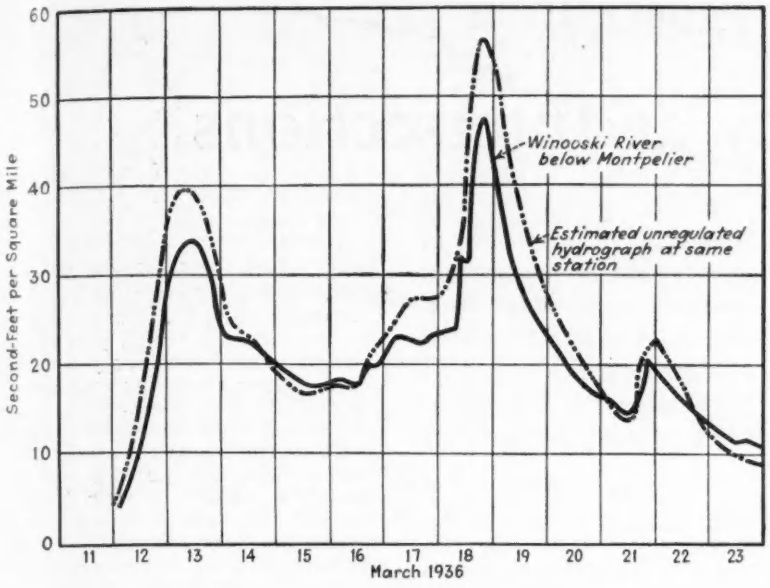


FIG. 7—HYDROGRAPH of the Winooski River below Montpelier showing actual runoff and the estimated runoff had the river been unregulated.

Inflow hydrographs in Fig. 5 were computed from measurements of storage and outflow. Because gaging of storage is not sensitive, actual peaks may have been 20 to 30 per cent greater than those shown.

Table II shows the peak discharges at points along the Winooski and its tributaries where known for both the flood of March, 1936, and November, 1927.

In its present stage of construction the Waterbury dam had little regulatory effect on the flood peak on that river.

Fig. 7 shows estimated hydrograph on the Winooski below Montpelier, had the East Barre and Wrightsville reservoirs not been constructed, and the actual flood hydrograph at the same point with these reservoirs functioning. As the unregulated hydrograph is based on inflow hydrographs to the two dams and on an estimated rate of flood travel, the peaks shown may be much less than what would have occurred had the flow been unregulated.

The two reservoirs functioned as planned and although their control, regulating only 109 sq.mi. of drainage area, was only a part of that contemplated in the ultimate plan of seven reservoirs (regulating 616 sq.mi.) their effect was noticeable and satisfactory.

The improvement of 15 mi. of channel below Montpelier must not be overlooked in considering the control effected. This control is not shown in discharge hydrographs, but it is shown in lessened damage and inundation. Without the two reservoirs holding back water the peak flow at Montpelier, Vt., would have been about 23,700 sec.-ft. or 40 per cent greater than bankful channel capacity. Without the reservoirs the city of Montpelier would, therefore, have again experienced water flowing through the streets, repeating the damages so frequent in the past. Inundation occurred within the city of Barre, Vt., from the Steven's Branch. This stream was unregulated, the offer of the federal government to construct a reservoir for this purpose having been rejected. Had the East Barre Dam not been in operation, the peak at Barre would have been about 55 per cent greater and widespread damage would have resulted.

Direction

The work is being carried out under the general direction of Col. E. D. Peck, division engineer, North Atlantic Division, by Lieut. Col. John W. N. Schulz, district engineer, First New York District. Major P. M. Ellman, is in local charge of the work.

Long Heavy Girders Placed with the Aid of Sand Jacks

TWO GIRDERS, 142 ft. 2 in. long, weighing 121 tons each, were recently lowered onto their pedestals by the use of sand jacks 12 ft. high, 3 ft. wide and 11 ft. long. The girders are used in a grade separation structure on Waukegan Road in Glenview, Ill., that will carry double tracks of the Chicago, Milwaukee & St. Paul R. R. over a four-lane highway at a skew of about 20 deg. The grade crossing that is eliminated has been responsible for thirty fatal accidents.

Since, because of the skew, the maximum deflection of one girder will occur opposite the end of the other, the roller castings used have a dome shape with a 2-ft. radius fitting into the socket of the upper casting with 2-ft. 4-in. radius, to permit slight movement in any direction. The sand jacks—boxes built of 8x16-in. timbers for the sides, with the vertical timber bulkhead ends tied by steel straps and rods—were set forward of these castings and the piers on a temporary cribbing made up of bridge floor steel. The girders arrived at the site on two special "well" cars with drop center separated by a third idler standard flat car, and were moved sideways into position over the sandjacks by the use of screwjacks.

The sandjacks are notable because of their large size and their method of operation. Bearing for the girder on the sandjacks was provided by a steel plate 20 in. wide supported by heavy timbers which rested on the sand. Workmen standing on top of the jacks removed the sand with small shovels, permitting the girder to sink. The timber sides and steel end straps of the jacks were removed as the jacks were emptied.

The work, which will cost $260,000, is being carried out with federal funds by the Illinois Highway Commission, Kendrick Harger, district engineer; R. M. Dillon, district construction engineer, and Martin Casey, resident engineer. The railroad company made the designs for the girders. William J. Howard & Co. are the steel erectors and sub-contractors for Paquette Engineering Corporation, general contractors for the bridge structure. Robert A. Black & Co. hold the contract for excavation and for road work.

GIRDER on car on temporary trestle ready to be jacked on greased skid plate over sand box at right and then lowered 12 ft. onto pier in right foreground.

CURRENT HIGHWAY PROBLEMS—II

Freeway at Intersections

Regulation of the flow of traffic through intersections designed to minimize interference and delays, to be most efficient, must call for orderly movements in directions taken naturally by the driver

By H. W. Giffin

Engineer of Surveys and Plans, New Jersey State Highway Department, Trenton, N. J.

BEGINNING a few years before the depression, highway travel speeds took a decided spurt upward. Density of travel had increased until main road capacity was becoming overtaxed and travel friction on the straightaways was multiplied many times on the intersections. A general movement to reduce intersectional friction was begun and its progress soon indicated an appreciable trend in highway practice. Activity in intersection improvement was most pronounced in the states and on the crossing roads that had the densest traffic; it is on the wide experience of one of these states that the following discussion and conclusions are based. —EDITOR.

PRESENT TRENDS of highway transport indicate that the use of the motor vehicle and its speed of operation will increase. In providing for heavy streams of traffic where they intersect practice has now progressed from the simple junction of two ribbons of pavement to the enlarged intersection, the traffic circle and the grade separation. The improved types have been in use for only a short time and to a very limited extent; the best methods of operation are not well understood by the majority of motorists, and the best principles of design are not too well settled in the minds of even the best-informed engineers. What to do in the future can, however, be learned from a study of what we have today, and improvement may be projected along the lines indicated by such a study.

Stop and go crossings

By and large the two-road crossing prevails. At such intersections vehicle paths cross and each path divides into three, if we count as we must, interchange by right and left turns from one road to another. Twelve paths are thus formed, eight diverging and merging, and four going straight through, making sixteen points of crossing. Conflicting crossings can be avoided only by some interruption of movement, and in present practice the stop and go sign and the traffic light have been resorted to for this purpose. Experienced drivers now pass through these stop and go intersections so as to avoid unnecessary conflict and delay by following paths chosen for their least resistance.

A driver intending to turn to the right maneuvers to a position on the extreme right of the roadway as he approaches the intersection. From here he can turn right directly on getting the signal, without having his path obstructed by those alongside or behind. A driver intending to turn to the left maneuvers to a position on the extreme left, at or near the center of the roadway. From this position he can, on getting the go signal, pass in front of oncoming straight traffic with little or no interference, although he may occasionally delay that traffic slightly. If he is late in getting into position, he can wait a moment and pass between or behind groups of approaching traffic. Those intending to go straight through take a position between those intending to turn off, and they line up abreast in the remaining space. When permitted to go ahead they move forward in parallel lanes across the intersection. The merging into the original number of lanes is governed by differences in speed and by roadway space available.

This natural distribution of traffic worked out by drivers themselves following their own inclinations in pursuing their desired courses with the least hazard and delay is as orderly as conflicting movements of this nature can be, and at the same time permits a reasonable freedom of action. Should not this method of operation be encouraged? Stop-and-go intersections can be improved by widening the approaches as shown by Fig. 1. This enables traffic to clear the intersections rapidly and increases its capacity; helps to do away with long lines of waiting cars, moving up only a short distance several times between light changes; provides a more balanced design between the roadway and its intersections with regard to traffic capacity, and postpones or eliminates the need for costlier intersections. An important problem for highway designers, traffic engineers and administrators of the future will be to develop this type of intersection to its limit of efficiency of operation.

Traffic circles

A type of intersection which has appealed more to the public than the flared type with signal regulation is the traffic circle. As in the flared type, all movements are made at grade, but they occur differently (Fig. 2). All traffic keeps to the right of a central plot, moving along in a continuous rotary movement on a one-way connecting road running in a counter-clockwise direction. Where constructed at the intersection of two or more straight roads, it is usually necessary for all traffic to turn somewhat sharply to the right to enter the intersection. This calls for reduced speed at which movement is safe on the one-way road between entrances and exists. When the circle is well designed, crossings are made at flat angles and, as all traffic is moving in the same direction, any damage from contact is usually slight if drivers are attentive to their driving. The right turn is ordinarily free from other movements if drivers do not swing into and out of the main stream, making mergings, crossings, and re-crossings which are unnecessary. The left turn is made by going three quarters of the way around the circle—each left turn crossing six other vehicle paths. Straight-through traffic crosses four other paths.

Some of the disadvantages of operating the traffic circle brought out by experience can be overcome by improved design, by driver education and by effective signing. The appearance of the approach to many circles is similar to that of a simple fork in the road. Drivers who trust their sense of direction rather than signs and who wish to make a left turn, are likely to turn to the left and run counter to the regular movement. Then, too, in the midnight hours many drivers come into the circle at excessive speed and are unable to travel on the sharp turns safely. Traffic circles are necessarily circular or oval in shape and it is difficult to keep one's sense of direction in traveling on a curved path because it is hard to gage the length of arc along which one has gone and to fix mentally one's bearings. Many drivers arrive at their exists unexpectedly and make a sudden turn—often into the path of other vehicles. Moreover, when passing through a traffic circle on rainy nights, each drop of rain on the car windows reflects all unshielded sources of light, and one has an uncomfortable feeling of being involuntarily revolved. Unequal angles of intersecting roads require unequal lengths of quadrants and often changing radii of curvature on the circle. Where the difference in curvature is large the change is obvious, but where the difference is smaller it may be imperceptible to the eye and the speed

which feels right on a flat curve may be dangerous on an adjacent sharper one.

However, even though there is some little confusion in the merging and segregation of traffic on circles, these disadvantages are more than compensated for by the continuity of movement in contrast to the stopping and starting at the ordinarly or flared intersection. Many of these disadvantages undoubtedly will be overcome for future traffic as the principles of design and the method of operation are better understood by engineers and by motorists.

Grade separations

The most popular and the most expensive type of highway intersection is the grade separation. Here operation of through traffic is practically the same as if no intersection existed. There are, however, some divergencies and mergings where there are connecting roads or ramps for the interchange of traffic from one roadway to the other. The number and arrangement of ramps depends upon the necessities and physical features at the site. One arrangement which provides rather complete provision for interchange without much interference is the so-called cloverleaf, in which either four two-way ramps or eight one-way ramps are provided (Fig. 3). The right turn is made in the usual way and often at relatively high speed—sometimes too high for safe and easy merging with main-stream traffic. The left turn is made after passing beyond the separation by making a three-quarter turn to the right. This is the reverse of the natural movement and is confusing to those habitually guided by their sense of direction rather than by signs. All left-turn

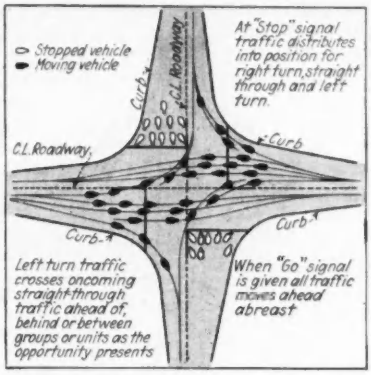

Stopped vehicle
Moving vehicle
Curb
C.L. Roadway
At "Stop" signal traffic distributes into position for right turn, straight through and left turn.
C.L. Roadway
Curb
Curb
Curb
Left turn traffic crosses oncoming straight-through traffic ahead of, behind or between groups or units as the opportunity presents
When "Go" signal is given all traffic moves ahead abreast

FIG. 1—STOP AND GO INTERSECTIONS can be opened up for safer and more free vehicle movements by widened and flared approaches.

traffic crosses the path of other left-turn traffic from two other directions. A vehicle from the south turning west crosses the path of one from the east turning south and that of one from the west turning east. These are all flat-angle same-direction crossings and the probability of contact is comparatively remote.

Modified types of crossing

In the separation of grades where the roads are not of the same importance, it is not always economical to provide as many connecting ramps as in the cloverleaf type. Sometimes only one or two ramps are provided and then some crossing of traffic paths is necessary. Several different arrangements have been used; the selection depends on the volume of traffic on each of the roads and on topographic and other conditions (Fig. 4). Left turns are sometimes made by leaving the main road on the right side and then turning left, crossing straight traffic on the road of lesser importance. In other cases the ramp arrangement is such that some of the turns are made as in the cloverleaf type. At some intersections part of the left turns may be made by one method and part by the other. While this may produce a little confusion at times it is not of great importance in comparison with the saving in cost of ramps.

There are also a few grade separation intersections involving rather elaborate arrangements of one-way roads. They are expensive and only heavy traffic justifies their cost. Their operation presents very little of a problem as they are practically self-operating. They do, however, require proper segregation as the divergencies at forks include large volumes of traffic.

Remaining problems

In the future all of these types of intersections and many variations and combinations of them will be needed on our heavily traveled highway systems. There is need for education in the best methods of operation, for aids to warn motorists of their existence and for means of advance differentiation. With more frequent use, motorists must learn to depend more upon signs than upon their sense of direction until the way is familiar to them.

The designer of the future will have to simplify complexities so that movements become more natural. This has been appreciated more in later designs where the natural movement is to some extent suggested by the layout, especially for the main traffic streams. Those of lesser importance numerically are correspondingly penalized as the main streams are favored, and how much to sacrifice safety and convenience in one movement in order to favor the greater number is often

FIG. 2—IN CIRCLE INTERSECTIONS all crossings of traffic are at flat angles with all vehicles headed in the same direction. Thus the collision risk is lessened.

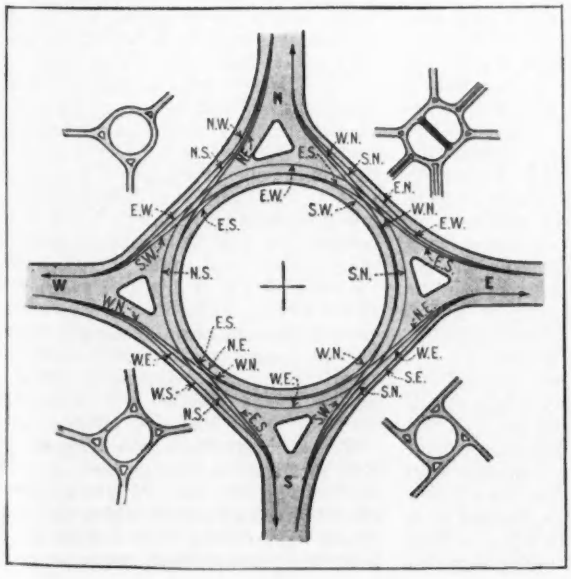
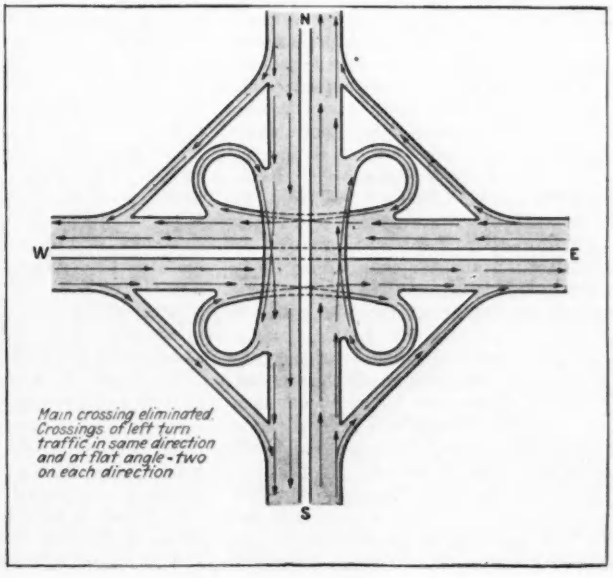


FIG. 3—GRADE SEPARATION with cloverleaf ramps allows virtually free through traffic and provides interchange by simple divergencies and mergings.



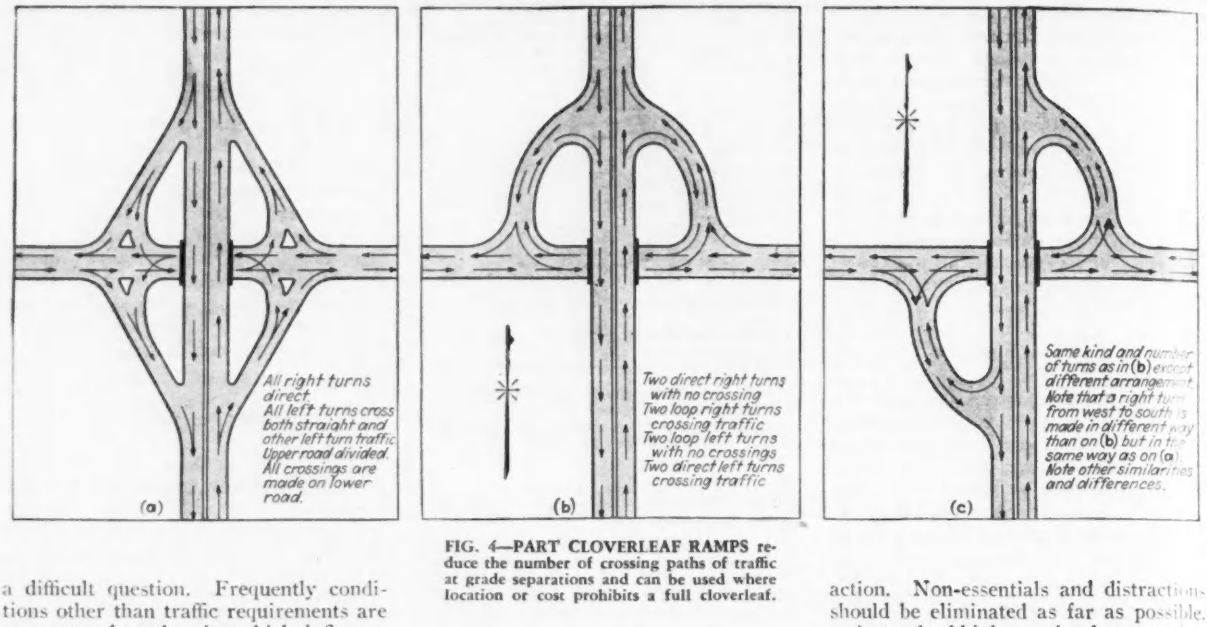

All right turns direct.
All left turns cross both straight and other left turn traffic.
Upper road divided.
All crossings are made on lower road.
(a)
Two direct right turns with no crossing.
Two loop right turns crossing traffic.
Two loop left turns with no crossings.
Two direct left turns crossing traffic.
(b)
Same kind and number of turns as in (b) except different arrangement. Note that a right turn from west to south is made in different way than on (b) but in the same way as on (a). Note other similarities and differences.
(c)

FIG. 4—PART CLOVERLEAF RAMPS reduce the number of crossing paths of traffic at grade separations and can be used where location or cost prohibits a full cloverleaf.

a difficult question. Frequently conditions other than traffic requirements are encountered at the site which influence the layout adversely, so that the designer is prevented from building an ideal intersection and even limited in adequately meeting traffic requirements. Each design must prove its economic justification and is, therefore, usually a compromise with the ideal. In the future a fuller knowledge of the relative worth of improved features of designs as against the cost will produce better designs for the expenditures made.

Traffic control essential

All of these types of intersections involve some regulation or regimentation of traffic into desired lanes or streams. The form of the layout is often sufficient to provide this, but in some cases channelizing islands are necessary to insure smoother operation and to prevent the reckless or ignorant driver from becoming disorderly in an otherwise orderly movement. The form and dimensions of these islands are often determined by trial in order that the most efficient operation can be secured. In this way the principles of island design can gradually be gleaned from experience in the operation of temporary or trial islands. Regimentation should follow the lines of movements which are natural and at the same time safe; it should be reasonable enough to win the support of a very large majority of drivers or it will be disregarded by enough of them to prove a detriment.

The fundamental principle of successful operation of all types of intersections is orderly movements based on advance knowledge, particularly where advance suggestion is essential to smooth and safe operation. For those who are inattentive to their location, those who through inexperience in driving do not yet know the best position for proceeding in the desired direction, those who are strangers in the locality and those who fail to locate themselves because of darkness, rain, fog, or snow, some special aids are essential to promote advance knowledge and orderly segregation. The aids in common use are signs, lights, reflectors, reflector buttons, and painted lines as traffic stripes.

Signs—The oldest method of giving aid to the traveler is by signs—dating back to the single post supporting many weathered signs. These faded monitors were doubtless adequate in the day when traveling by highway was very restricted both in volume and radius of travel; but the present need is for signs which work full time in all conditions of weather and which may be read and understood at prevailing speeds. Brevity and spacing are of the utmost importance. Symbols should be substituted for words whenever symbols can convey the message as clearly. Especial care should be given to the design of each sign so that the information on it impresses the driver. Thought should be given to the distributions af signs, as a large number in one location is distracting and confuses the driver. Each sign should be a complete unit in itself to effectively convey a message. Spacing should be regulated by the time necessary for grasping each message before the message on another sign is visible. The order should be general information first and then specific information in the order of its importance as affecting the greater number.

There are many distractions to drivers at intersections and many things are in competition for their attention. A few effective signs, properly spaced, are more enlightening than a whole directory no matter where placed or how distributed. The object is to inject a thought or a series of thoughts into the driver's mind and to make them stick and govern his action. Non-essentials and distractions should be eliminated as far as possible.

A standard highway sign language has been urged and ultimately will probably be adopted, but before this is done there should be further development of effective signs. Few are serving their purpose as well as they could and the adoption of standards at this time would tend to freeze an immature development. Much more progress should be made before sign erection and designing can be allowed to become merely a matter of routine.

Lights—All of the special intersections mentioned above should, where possible, be illuminated in such a way that the source of light is not an additional distraction. The outlines of the intersections should be so built as to reflect this illumination and stand out in all kinds of weather. In snow the outline should be suggested either by the planting of shrubbery, by painted posts, shielded reflectors, or reflector buttons. The lighting of signs which are essential for proper movements should be distinguished from that of direction signs giving names of places. Drivers are interested in direction signs but should be more interested in their driving conduct at an intersection. It is better for a driver to get lost than to endanger the safety of himself or others. The selection of lighting fixtures and their location is largely a matter of consultation with illuminating engineers of equipment and utility companies in the initial stages of this work. If much of the work is to be done it pays to develop a highway engineer for this work, as a continuing policy; experience in the results and a closer sympathy with the purposes to be attained will in the end better serve the motoring public.

Reflector Buttons—Improvement in reflector buttons has opened a field for their greater use. Where properly spaced on a background which does not reflect light readily these buttons form a marked contrast which can be seen for

several hundred feet beyond the seeing range of headlights. When used to form letters on signs the effectiveness of the sign at night is often greater than in the daytime as the presence of the buttons detracts from the sharpness and visibility of the painted letters. Any reflector requires a strong source of light to be effective. Parking lights give poor or negligible results and many motorists habitually use parking lights, especially at dusk.

Design is a specialty

In conclusion, the operation of all special-type intersections is so different from that part of the roadway between intersections, and the development of these types is so recent that the design of the intersection itself and the aids for its successful operation are special phases of highway engineering to which intensive study must be given in the future. The responsibility of highway administrators must go beyond the perfunctory erection of signs and lights. While no guarantee of a safe passage can be given to even the prudent, every reasonable precaution should be taken to insure that each driver is apprised of what he is to encounter just ahead of him. An intersection isn't any longer just a junction of roads: it is a means of continuing a journey safely and conveniently. A sign isn't just a sign; it is a message to each driver, suggesting a correct conduct amid distractions and conflicts. All require most careful study.

In the old horse and buggy days it was not so necessary to be constantly alert. Many delinquencies of the driver could be overlooked as they were often take care of by the horse. The modern motor vehicle, however, has supplied an excessive amount of destructive power without the small but saving amount of horse sense. Motor vehicles do not drive themselves; they require attentive and efficient human machines in control. If there were only reasonable restrictions as to who may drive an automobile a great improvement in safety and convenience would follow. But only an accident-conscious public would approve of drastic but necessary measures to cull out the few drivers who give most of the trouble and to require a certain minimum of operating knowledge by the remainder as a requisite for the privilege of driving. In the meantime, the application of patience, ingenuity and intelligent analysis of driving habits and driving psychology by the highway designer and a generous supply of funds by the public will make the operation of intersections as safe and convenient as the inconsistencies of human nature will permit.

Public Works Conference

Toronto meeting sees ratification of merger plan uniting municipal engineers and officials in a single society whose aim will be the improvement of public service administration

FOLLOWING two years of operation under a joint secretariat the 43-year old American Society of Municipal Engineers and the 17-year old International Association of Public Works Officials, meeting in Toronto, Ont., from Sept. 28 to Oct. 1, concluded arrangements through their respective governing boards to effect a merger and form the American Public Works Association. Formation of the new body will be effective Jan. 1, 1937, if the proposal receives affirmative vote in the letter ballot which the executive director was ordered to submit to the 800 members. The principal purpose of the newly organized A.P.W.A. is stated to be "the advancement of the theory and practice of the design, construction, maintenance, administration and operation of public works facilities and services." Active members are to hold a public works office or shall be an officer, executive, staff member, or consultant to a public works body.

Technical program varied

The four-day session, attended by more than 300 representatives from the United States and Canada, reflected the aims of the new organization in its program. After opening with a symposium on public works problems, the conference then concerned itself with technical committee activities and progress reports on two manuals undertaken during the last year, one on street cleaning and the other on refuse collection and disposal. Administration claimed attention in six papers, varying in subject matter from the Toronto public works department to water and sewage plant management and personnel training. Paving, which formerly occupied practically all of the time of the A.M.E., still had a prominent position in a careful résumé of recent developments, soil stabilization, and low cost pavements. The twin subjects of housing and planning and lighting and traffic control received deserved attention.

The symposium on public works problems, in which a score of municipal administrators representing large and small cities took part, gave clear statement of the false economy in continued curtailment of public works budgetary appropriations. G. R. Byrum, street commissioner in Birmingham, Ala., reported that when recently his mechanical cleaning equipment became completely worn out after years of service, he could obtain no funds for replacement, thus making it necessary for him to revert to old-fashioned and expensive hand cleaning methods. Henry Cates, chief of the sanitary department at Atlanta, Ga., deplored the fact that funds for essential city services should be the ready target for all aims at economy. Since 1930 he has been forced to operate the garbage collection service with 20 per cent less equipment than normally required, and besides all regular street cleaning equipment was retired from active duty, thus placing an additional burden on already overtaxed facilities.

Snow removal and street maintenance problems were given considerable attention. Southern cities like Birmingham, Ala., and Atlanta, Ga., had unprecedented snow troubles this past winter; in the former, an 11-in. snowfall coupled with extremely cold weather for that locality sorely pressed the city authorities who were without adequate mechanical equipment to cope with the situation. In Atlanta, an 8-in. wet snowfall caused the breakage of tree limbs and more than 10,000 loads of debris had to be removed before the streets were clean. E. F. Martin, engineer of Montclair, N. J., stated that $50,000 has been the average yearly cost for removing snow in his town, yet during the past few years only $4,000 has been allowed in the budget—this past year he had spent $15,000 up to the time the new budget was approved, and as a result deficits have been incurred which necessarily will have to be carried in future budgets.

Improving dirt streets

City streets in many places were stated to be in bad condition due to the continued narrow policy of restricting funds to provide proper maintenance, and it was pointed out that this will

eventually exact an expensive toll by necessitating complete rebuilding. The need for immediate improvement of dirt streets by inexpensive hard surfacing treatment or semi-permanent methods of construction is another matter causing some concern. In Pittsburgh, for instance, there are 500 miles of unimproved streets on grades as steep as 25 per cent, and water erosion is not only creating a public hazard but in addition is causing severe clogging of sewers, according to William B. Shafer, superintendent of the bureau of highways and sewers. Lack of funds in St. Paul, Minn., has led George M. Shepard, city engineer, to adopt the following procedure in maintaining his dirt roads: the surface is first scarified with heavy discs, then covered with oil and a light application of stone chips. This has proven to be an economical and effective method of improvement.

Land acquisition to provide wider streets for expediting traffic is a problem of increasing magnitude in the East. Rochester, N. Y. has only one main thoroughfare through the congested business district, and to improve this condition said Thomas Morrison, commissioner of the public works, it would cost $12,000,000 of which $9,000,000 represents land cost. W. E. Rosengarten, engineer of Lower Merion (Pa.) township, cited the difficulty of obtaining adequate rights-of-way to improve "horse-and-buggy type" village streets.

The design and construction of flood control projects has complicated the work of the city engineer in New York State communities according to reports made by Harry Eustance, of Ithaca, and Morgan Hayes, of Rochester. With these new problems attached to the multiplicity of diverse duties with which the municipal engineer is already burdened, F. R. Storrer, engineer of Dearborn, Mich., pointed out that it is increasingly important that engineers engage in wider exchange of ideas at meetings and through the facilities of the engineering press.

Technical committee reports

In a preliminary report on the status of city and regional planning, Thomas Buckley, of Philadelphia, outlined the various planning groups that now overlay practically the entire United States. Topping the list is the National Resources Committee followed by four super-regional organizations, namely, the Tennessee Valley Authority, the Pacific Northwest Region, the Rocky Mountain Region, and the New England Region. There are six metropolitan regions with centers at New York, Chicago, Philadelphia, St. Louis, Kansas City and Washington, and 46 state planning boards. (Delaware and Louisiana are the states without such organizations.) In addition to these there are at least 1000 county, city and town planning bodies and numerous boards and commissions functioning in connection with the preservation and development of such resources as minerals, forests, water, agriculture, and industry.

The report dealing with current research on paving materials listed all those schools and organizations which are engaged at the present time with investigations on street paving, design, construction and maintenance.

According to the technical committee reporting on municipal field engineering, only 47.1 per cent of the United States is topographically mapped, and at the present rate of progress—last year it was 0.4 of 1 per cent—it will require more than 130 years to complete a country-wide survey. While there was only nominal progress in general ground surveying during the past year, there was noted a definite trend toward wider utilization of aerial surveying methods. In discussion it was brought out that aerial photography for use in large scale mapping operations, such as required for city maps, is not accurate; distortions caused by aberration errors and enlargement faults have been found to be extremely difficult to correlate and correct.

In a paper entitled "Airports and Their Relation to National Defense," by R. F. Hedtler, chairman of the airport landing field committee, the necessity of providing for more landing fields in this country than we now have was stressed. The estimate of cost to meet this requirement is $120,000,000. Some $30,000,000 has been allocated and plans are under way to release $40,000,000 within the year.

Traffic control

Indictments were leveled by Burton W. Marsh, chairman, committee on traffic control, at the engineer for abetting traffic problems by his failure to give proper study and attention to this matter. As evidence of this he recited that only 25 engineers replied to a questionnaire sent to 252 members asking what progress had been made in their cities toward acceptance of suggestions made on last year's report under the heading "First engineering steps to improve the traffic situation." Those who did answer, mostly traffic engineers, indicated that road marking, physical improvements, signs and signals had been given the most attention. Traffic surveys were the next most prominent item cared for. Lack of interest in accident records was particularly significant since, so Mr. Marsh stated, this is a logical engineering type of activity of fundamental importance. To arouse interest it was suggested that local chapters develop discussions on the subject.

The street lighting committee, Paul H. Goodell, chairman, stressed the fact that two-thirds of all the fatal and serious road accidents are resulting from one-fifth of the total traffic. This one-fifth is that which uses the thoroughfares during the night. If adequate lighting were in effect, it was stated to be reasonable to expect a saving of 3,500 lives annually. While many states have no legislation concerning the lighting of highways, 22 miles of improved lighting were recently installed on highways in New York and similar installations are being made in a dozen other states. Instead of serving only an ornamental purpose, lighting is now looked on as a prime factor in promoting safety.

Industrial wastes

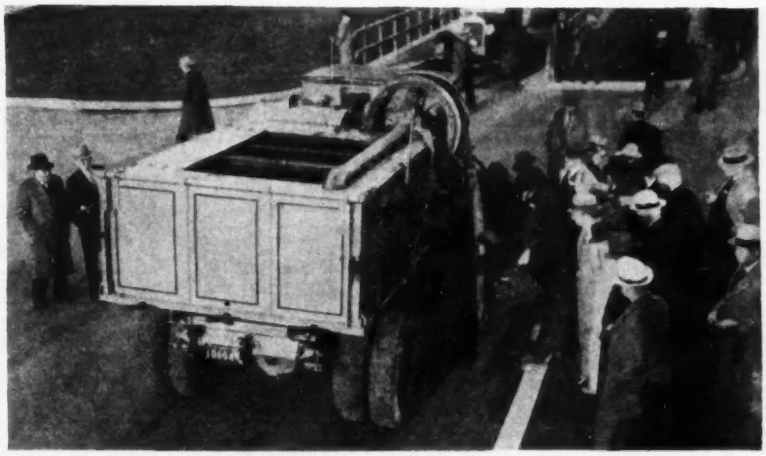
In his report on the relation of industrial wastes to sewerage problems, Darwin W. Townsend, consulting engineer, Milwaukee, Wis., chairman of the committee on sewerage and sanitation, stated that industrial waste in this connection should be studied: (1) from the physical, chemical, biological, and germicidal standpoint, and (2) as to their effect on the carrier system and treatment works or process. Following a recital of various laws and regulations by municipal authorities to safeguard sewer systems and treatment processes, the report outlined general standards serving in Milwaukee as substantial guides relative to waste admission to or removal from the sewer system. These regulations prohibit entry of the following: (1) any solids that might tend to form deposits in sewers; (2) wastes containing active chemical constituents which through their activity might tend toward destruction of construction materials and equipment or be deleterious to treatment processes; (3) free oils, greases, tar and resinous substances; (4) wastes of gaseous or inflammable character which may through their presence result in injury or death to men engaged in work around sewers; (5) all uncontaminated waters such as condensing water, cooling water, yard and roof drainage water; these must be diverted to storm water sewers or to natural water courses.

The various methods which have been effective in pre-treatment of industrial wastes before their entry into sewer systems were summarized by Mr. Townsend. Settling tanks and screens, alone or in combination, were recommended in cases where removal of heavy suspensions and fibrous matters was desired. In dealing with wastes containing acids, such as pickle liquor from steel plants, discharges from engraving plants, metal plating work, storage battery plants and allied industries, it was suggested that various methods of neutralization could be adopted. The tarry wastes from producer gas plants and coke and gas works have been found to be susceptible to satisfactory pre-treatment through precipitation of the tar-laden waters in settling tanks followed by filtration of the effluent through coke breeze filters.

Packing houses and allied industries, such as sausage manufacturing establishments, are producers of substantial quantities of oily, fatty and greasy wastes. The removal of these substances is essential before the liquid wastes are discharged into the sewers.

not only because of the unwelcome presence of this material in a treatment plant, but also because of the tendency for it to adhere to sewer walls, thus reducing the carrying capacity of the sewer, sometimes to a point of complete stoppage. The installation of adequately proportioned grease traps not only results in effectively keeping the greasy substances out of the sewer system, but may be a source of revenue to the plant owners, who find a ready market for the grease thus reclaimed.

A matter of considerable importance is that of the pre-treatment of sulphite waste liquors produced by pulp mills in which biochemical oxygen demands frequently exceed 20,000 p.p.m. The discharge of a waste possessing such an intense affinity for oxygen into a public sewer system for treatment in a biological sewage plant cannot be justified.



DELEGATES at the public works meeting absorbed in a demonstration of truck-mounted eductor equipment used by the Toronto department of works for vacuum removal of debris from catch basins.

Manual committees

Progress reports by committees charged with the preparation of manuals of practice on street cleaning and refuse collection and disposal respectively, indicate that within the next year both of these important reference works will be completed. An outline of the scope of the refuse manual was submitted by Carl Schneider, chairman, who stated that further research work was required to obtain necessary analytical data. One of the features to be included is the determination of current trends and practice to be analyzed by means of a questionnaire survey. Among the major topics to be considered are: administrative methods, definitions and descriptions of waste materials, collection facilities, disposal methods, and European practices. The committee on the preparation of the street cleaning manual, announced an outline of the contents.

Toronto department of public works

Despite the absence of civil service protection, long-time tenure of office for civic employees is the rule in Toronto. Here technical fitness is the only qualification governing appointments because of a strictly non-partisan municipal government, according to R. C. Harris, commissioner of works. The department of works is organized with eight sections for administration and operation, each of which is under the direction of an executive head who has reached his present office by promotion after an extended period of service. The operating sections, comprising roadways, sewers, railways and bridges, water supply and water distribution are in charge of engineers, each of whom has a complete organization for the handling of his work. There is also the head office, a coordinating department headed by the city engineer, who is also the commissioner of public works, in addition to departments of accounting and photography.

One of the newer developments in improving the functioning of municipal activities in Toronto is the formation, in 1933, of a public utility coordinating committee, consisting of representatives of the engineering departments of the gas, electric and telephone companies who meet with engineers of the department of public works to make recommendations and report on matters pertaining to street surface and underground construction. In this way it is possible to plan and correlate all street construction and developments in which utility services and municipal structures are involved.

The water supply section of the department is now supervising the construction of additions and extensions to the waterworks system which will ultimately cost $1-,000,000. The new system, comprising an intake tunnel from Lake Ontario, a distribution tunnel, and a pumping and purification plant, will duplicate, and can be operated separately from the existing plant.

Paving progress

Soil stabilization under pavements and other less recent developments in concrete slab design were well outlined in a comprehensive paper by H. F. Clemmer, engineer of materials, department of highways, Washington, D. C. Street pavement designs now reflect the thickened edge requirements of highway slabs as well as the elaborate doweling and expansion joint provisions. Mr. Clemmer advocated richer mixes for bases under brick and asphalt surfaces with predetermined joint location. Base construction for sub-bases requiring stabilization was brought forward. The lessons of the highway research were thus fitted to city street requirements. Specifications on stabilization and the richer bases are soon to be forthcoming.

George Sandenburgh, city engineer, Ann Arbor, Mich., in his paper on "Low Cost Pavements for Cities," outlined the experience of his city since 1927, when treatment of unpaved streets to eliminate the dust nuisance started him in an oiling program. This has now developed into a road mix surface treatment costing $.25 per sq.yd. utilizing city equipment costing $15,000. The first of these payments is nine years old and he estimates the life of strictly residential streets at 10 to 15 years. In discussion, G. H. Stanley, city engineer, Burlington, Vt., quoted $.50 per yard for similar 1½-in. surface plus a 4½-in. gravel base stabilized with emulsified asphalt.

Public service training

Training engineers for the field of public service is a matter that has not received proper attention in our colleges, according to Louis Mitchell, dean of engineering at Syracuse University, who made suggestions regarding the type of curriculum that should be adopted. Judging from replies received from a survey among older graduates, it was indicated that too much emphasis is being laid on specialized technical courses to the neglect of the broader aspects of training for the administrative leadership. Public speaking, English, economics, and finance courses should be given more attention.

Advanced study, preferably after a few years of practical experience, was held to be a worthy objective by Dean Mitchell. He explained that the subjects that would be most profitable came under the classification of economics and political science covering such matters as legal, personnel and health problems.

Slum costs to municipalities are greatly in excess of the taxes collected, stated F. J. C. Dresser, district man-

ager, U. S. Housing Division, Cleveland, Ohio. Citing a 333-acre slum area in Cleveland, he said that the net cost of direct services over taxes was $728,000, and the net excess cost of education was $271,000. Though but 2½ per cent of the population of the city lives in this area, 14.4 per cent of the total fire protection cost was spent here, 6.5 per cent for police protection, and 8 per cent for relief. Cleveland puts $1,750,000 more money into the area than it receives in the taxes collected. From this premise he quoted a total saving of $637,000 that could be made by replanning and rebuilding the area, eliminating needless streets, cutting down street cleaning and lighting and garbage collection.

On replanning of cities the following aim was set forth as indicative of modern thinking: (1) the mass continuity of cities should be broken up; (2) the gridiron layout so prevalent in our cities with 30 to 38 per cent of the land in streets and alleys can and should be reduced to 15 per cent thus making it possible to convert the surplus into open spaces for recreation; (3) residential areas must be kept away from arterial highways in the interests of safety.

Cross connections

In a paper dealing with the enforcement of cross connection regulations, J. Arthur Jensen, engineer, waterworks department, Minneapolis, Minn., called attention to the new dangers from the greatly extended use of groundwater for air conditioning. The principal force to be applied to eliminate cross connections is the power to deprive premises of water until service arrangements meet regulations. Regular inspection is essential. Separate systems are necessary if the consumer uses well water for air conditioning and public water for boilers, hot water and general plumbing use. More stringent enforcement in the future is expected when its importance as a health safety measure is recognized.

Other papers concerned with water department administration were presented by Hal F. Smith, department of water supply, Detroit, Mich., who discussed billing and customer accounting, and by R. L. Dobbin, general manager of the Peterborough, Ont., Utilities Commission, who spoke on effective use of operating records.

Sewage plant administration

Management of a sewage treatment plant is a cooperative activity in which both state and municipal interests are correlated in order to obtain maximum service in return for minimum expenditures, according to Frank W. Jones, consulting engineer of Cleveland, Ohio. The state government in its concern for the abatement of stream pollution and the maintenance of public health over a wide area demands adequate municipal sewage treatment measures and to that end has injected itself into the management of these facilities. The influence of the state has been directed toward increasing knowledge through educational conferences among plant operators, and it has done much to improve plant efficiencies and processes through cooperative supervision.

Present day municipal officials, contrasted with those of former years, are more keenly aware of the importance of sewage disposal and their responsibility for maintaining treatment plants at peak efficiency. To this end there is greater appreciation of the need for employing competent personnel who can be assured of tenure regardless of political changes. It was pointed out by Mr. Jones that plant supervisors and operators occupy the rôle of administrators of a vital municipal activity in which not only technical skill but the application of competent business principles represents the essential elements in the efficient discharge of their responsibilities.

Garbage grinding summarized

In a paper on garbage grinding, Mark B. Owen, president of the International Association of Public Works Officials, expressed the opinion that the application of the grinding principle appears to present the first step in a method of disposal that may become the common and accepted practice in handling municipal garbage via sewer systems. Although adoption of this method will throw an increased burden on sewage treatment works, it was stated that the biggest problem, namely, the disposal of solids, could undoubtedly be solved by the use of sludge incineration.

Application of the grinding plan of disposal can be made in three ways: (1) by construction of grinding stations over large sewers and centrally located sections of a city; (2) by grinding equipment at the sewage plant site; and (3) by installation of home grinders attached to kitchen sink drains.

The central grinding station scheme was first tried at Lebanon, Pa., in 1923, and the next use of the idea was made in Baltimore in 1933 where 70 tons of garbage per day was disposed of in a 24-in. city sewer when it became necessary to relieve summer peak loads on the incinerator. Tests at the sewage plant showed that suspended solids were increased from the five-year average of 234 p.p.m. to an average to 271 p.p.m.; the five-day B.O.D. test showed only a slight increase but it was observed that "garbage solids are generally far from being in the same putrified condition as sewage solids," and that a 40 or 50-day incubation period might be necessary to show the true increase in oxygen demand. In 1934 the first municipal grinding plant for disposal of all garbage collected by city forces was built in St. Louis, Mo., and it has been in continuous service since February, 1935. The plant is located in the geographical center of the city and about 40,000 tons of garbage annually is discharged into the sewer system which empties into the Mississippi River. The disposal cost is about 50 cents per ton.

A grinder was installed at the grit chambers of the Indianapolis, Ind., sewage plant in May, 1935, as an emergency measure for garbage disposal when the city reduction plant was out of service for seven months. A total of 17,256 tons of garbage was added to the treatment plant during this period, and there was no observed disturbance of the activated-sludge process. Sludge production was only slightly higher than normal.

Grinding equipment designed to fit standard types of kitchen sinks is now available but their general use is restricted because of the present high price per unit. Normal household wastes are reduced to a fine pulp in this machine and then discharged to the sewer through a regulation plumbing trap into the usual drain line and then to the sewer.

It was suggested that an economical and feasible scheme for reducing garbage disposal costs would be the combined incineration of sewage sludge and ground garbage. This could be accomplished by discharging the ground garbage into a sludge mixing tank prior to dewatering by vacuum filters. At present there are nine sewage plants in the United States (one built and eight under contract for construction) which will be equipped to take advantage of this plan.

Refuse disposal at Toronto

Refuse disposal by means of incineration has long been approved practice at Toronto, according to J. A. Burnett, engineer in the department of street cleaning, who pointed out that the first incinerators in this city were placed in operation 46 years ago. That installation, comprising two small natural draft furnaces with five coal grates where additional heat was supplied to assist the burning of garbage, handled about 140 cu.yd. of waste a day emanating from a contributing population of 150,000.

A high temperature mixed refuse destructor of 180 tons per 24-hour capacity was placed in operation in 1917 and it represented at that time the most up-to-date installation of its kind on the American continent. Prior to that time comparatively little had been done with high temperature incineration but now the process is firmly established. In 1924 a 400-ton per day capacity incinerator was added to the city's disposal facilities. To meet increased demands another high temperature type unit was placed in operation in 1934. This latter is of 500 tons capacity.

Like the others it is of the continuous-grate regenerative type, with four cells to each of four furnaces provided with preheated forced air draft. The charging hole doors are controlled by compressed air mechanisms which are operated by the stoker men on the furnace floor. Material entering the furnace first drops on a drying hearth in the back of

the cell from which it is manually drawn over and spread on the grate where incineration is effected. The residue is drawn off the grate through large clinkering doors at the front of the furnace and passes down to the floor below into side dump ash trucks.

It was stated by Mr. Burnett that the layout of the furnaces in this last plant represented departure from conventional design. It consists of four furnaces, set up in pairs, back to back—this permits a stoking aisle opposite each unit, with four furnaces set in line.

Veteran awards for long-term city service was presented to R. C. Harris, commissioner of works in the continuous employ of Toronto since 1899, to Frank L. Thomas, surveyor, with Philadelphia since 1892, and to Joseph F. Delaney, surveyor and regulator, with Philadelphia since 1900.

The following officers of each organization were elected to carry on until January 1: for A.M.E., president, Thomas Buckley; vice-presidents, Guy Brown, Henry L. Howe, and F. T. Paul; for I.A.P.W.O., president, J. E. Root; vice-presidents, J. F. Flockhart, F. T. Paul, and H. L. Howe.

The Nuevo Laredo-Mexico City Highway

By Robert B. Brooks
Consulting Engineer
St. Louis, Missouri

AT PRESENT the national highway system of Mexico consists of two trunk roads. Caminos Nacional No. 1 runs 1,052 miles from Nuevo Laredo, through Mexico City, to Acapulo. The 770-mile section from Laredo to Mexico City was officially opened on July 1. Highway No. 2 2,557 miles from Nogales, through Mexico City, to Suchiate, is now under construction. There are 14,000 men working on this highway, mostly between Nogales and Mexico City.

The new Laredo-Mexico City section was built at a cost of about $17,000,000. It is of oil-treated macadam and gravel construction, with a paved surface 19 ft. 7 in. wide; out to out of shoulder is 32 ft. 9½ in. It is a full width road over its entire length, with widening at the sharper curves in the mountains. While there are no guard rails on the outside of curves, there are large stones painted white at short intervals. The road is patrolled over its entire length by motorcycle men who are expert mechanics and who can speak some English.

The writer was a member of a party of government officials, engineers, and representatives of automobile clubs and automobile dealers invited by the Mexican government to make the trip from Nuevo Laredo to Mexico City, as part of the opening ceremony.

The cavalcade left Nuevo Laredo for Monterey on July 1, headed by a motorcycle squadron and followed by service cars for mail, telegraph, and radio. The distance from Laredo to Monterrey is 146 miles. The road is excellently paved, running through an uninhabited country of cactus and chaparral. About half way out we began an easy climb of the foothills of the Sierra Mountains. Mamulique Pass is crossed at an altitude of 2,280 ft. and about 100 miles south of the border. The road dips down from this height about 1,000 ft., gradually coming back up to an elevation of 2,000 ft. at Monterrey. There are several excellent hotels in that city.

From Monterrey to Victoria the distance is 179 miles. The road is in good condition. For a distance of about 20 miles we ran through Huajuco Canyon, then through hilly country with an occasional orange grove. In the afternoon we went 85 miles farther to Villa Juarez. The highway runs through high tableland covered with brush.

About 110 miles out of Villa Juarez we passed through the town of Tamazunchale (pronounced the same as Thomas and Charlie). We were told that before the building of the road the people of this section had never seen an automobile. Tamazunchale is at an altitude of 500 ft. In a distance of about 20 miles from Tamazunchale to Chapulhuacan we climbed to almost 3,000 ft.

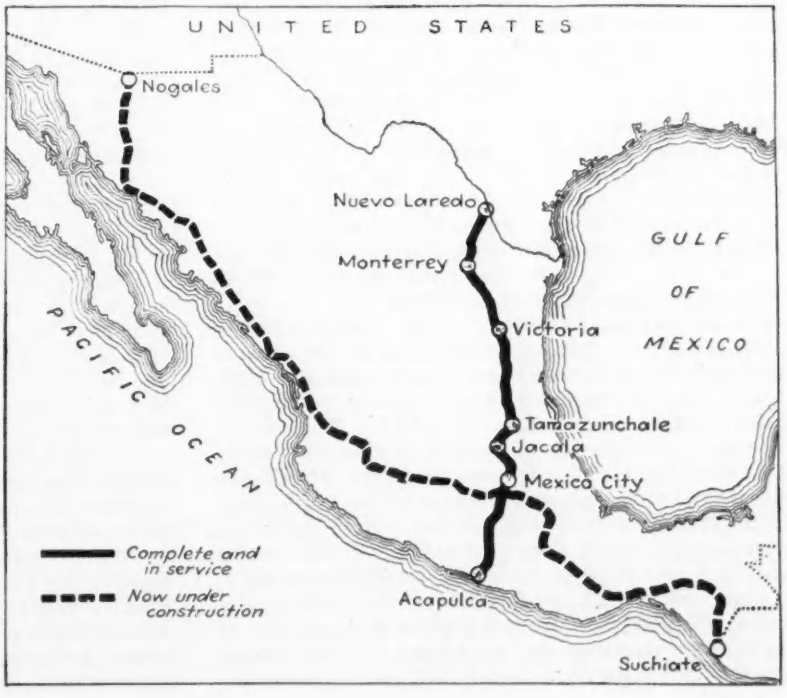
The 60-mile section of highway between Tamazunchale and Jacala is the stretch on which it has been reported that occasional slides were delaying traffic. At the time of our visit on July 3 the road had been completely widened and was in excellent shape. We drove through a torrential rain, which would certainly have developed any softness in the road.

At noon on July 4 the party reached the highest point on the route between Colonia and Actopan, an elevation of 8,300 ft.

Mexico City was reached on the evening of July 4.

The new highway is a section of the proposed inter-American highway. One thousand seven hundred and twenty-eight miles of the Inter-American highway lie in Mexico. The six-year plan of the Mexican Government provides that this section shall be completed by 1940, at a total cost of about $40,000,000. With construction of bridges over the main rivers in the Central American countries going ahead so rapidly, together with the fact that all of these governments have become highly road conscious, I am of the opinion that the entire 3,250 miles from Laredo to Panama City will be completed by 1940.

MEXICAN FEDERAL HIGHWAYS consist of a completed road from Nuevo Laredo to Acapulco and a route under construction from Nogales to Suchiate.



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Higher Public-Works Service

THE MERGER of two old-established organizations in the public-works field as the American Public Works Association may be hailed as a desirable and progressive move. For almost half a century the American Society of Municipal Engineers, older of the two merging groups, has labored for the advancement of technical knowledge. Alongside it, but much younger, grew up the International Association of Public Works Officials, devoted to improving public-works maintenance and operation. During recent years the need for a strong professional association interested in the promotion of improved practices in all phases of the public works field has become increasingly evident. Municipal services have expanded in type and scope, and new relationships have developed between state and federal agencies, all of which means that the administration of public works has been raised to a much higher plane. With it has come the concept of career possibilities in public-works service. The newly formed American Public Works Association, by unifying and expanding the objectives of its parent organizations, proposes to serve the present uncoordinated field of public-works service as to both technique and administrative method. In this undertaking *Engineering News-Record* offers welcome to the new association and bespeaks for it hearty support.

Reservoirs, Not Ponds

THE TWO COMPLETED flood detention reservoirs of the seven proposed for the Winooski River in northern Vermont caused a pronounced reduction in floods on the main stem of the river last spring, in sharp contrast to what happened in that river valley in the flood of 1927. Thus the Winooski River work stands as the symbol of what is to be expected from the program of flood control launched by Congress at its past session. But the work also stands as a warning that flood planning to be effective must be sound hydrologically. This was emphasized by Major General Edward M. Markham, Chief of Engineers, in speaking at the Upstream Engineering Conference a fortnight ago, a conference that was called for the purpose of drawing attention to the relation of headwaters to land and water conservation. General Markham took the Winooski work as an example of what could be done in reducing flood damage, but he went on to say that what was there accomplished was made possible only by relatively large detention reservoirs—that it could not have been accomplished at all with a lot of little ponds way up on all the tributary brooks. Others at the conference emphasized the same point. Ponds may have a place in water conservation as a means of increasing the underground supplies, but they must not be looked upon as a means of flood prevention. Floods can be controlled only by large reservoirs farther downstream, so placed and of such size that they can hold back a large part of the flood flow from the whole drainage area of the tributary on which they are located. That fact must be recognized in the flood-control work about to be undertaken if money is not to be squandered on headwater storage that can never be effective.

Structural Achievement

APPRAISAL of the contribution that the huge towers of the Golden Gate suspension bridge make to the structural engineering art is bound to take account of more than one fact. The towers are notable structures from almost any viewpoint. In size, they are taller and heavier than any of their predecessors; the maximum section of each leg contains the almost unbelievable total of 12,400 sq.in. of steel, arranged in a compact grouping of plate-and-angle cells. In appearance, they introduce a new pattern of bridge architecture, whose simplicity is aided by the omission of diagonal bracing above the roadway. By virtue of this very simplicity, however, the designers were confronted with the paradox of complication in analysis. That they not only were able to overcome the analytical difficulties but succeeded in developing a simplified direct method of analysis that offers possibilities of eliminating the paradox is among their important contributions. In brief, the method (first described in these pages by Prof. C. A. Ellis, *ENR*, April 26, 1934, p. 534) involves the solution of a number of simultaneous equations yielded by Williot diagrams that give the geometrical relations between the distortions of the various members of the frame. The boldness and architectural merit of the towers remains individual, but the design procedure that they engendered is a lasting asset of the structural art.

Construction Trends Upward

A NOTEWORTHY DEVELOPMENT is revealed by current figures for construction contract lettings. The new activity that set in during 1935 was sharply accelerated three months ago, with the result that the construction record has climbed to its highest point in five years and now stands at better than half the 1927-29 level. The total volume reported to *Engineering News-Record* is up 71 per cent over last year, which is far ahead of the best expectations of last January. Evidently deferred renewal demand is at last making its pressure felt. This fact, together with the inevitable need for expansion in many fields, suggests that construction volume will within a few years go beyond past records—unless the skilled labor shortages already felt in many places check its growth. The hoped-for normalcy will not be reached, however, until private construction accounts for its two-thirds of total construction and until the financial burden of public works is shifted back from the federal government to the states and cities. The nine-month period ended with an accelerated trend in both directions: private work has accounted for nearly two-thirds of the contracts of the past few weeks, city tax collections are phenomenally improved, and municipal credit is better than it has been in years. The cities are getting back on their feet and have only to reassume the responsibilities that they cast off during trying times to carry us farther toward restored construction activity. For the present the sensational increase in construction volume means two important things—that industrial recovery is entering its final chapter, and that heavy-goods production will receive a further impulse from renewed construction, its best market. Finally, these events suggest that the period of low-price construction may be drawing to a close. Next year is sure to see an even greater volume of building, but it is likely to be at higher cost levels.

Bad Building

OLD AS HISTORY, the question of bad building is always with us. It gave rise to punitive law codes in most ancient times, and it continues to be a problem that not only troubles the present day home-seeker and home owner but goes right into the soundness of our whole financing and investment system. Mortgage defaults, mass foreclosures and the creation of rotting city districts are among its consequences. Recent complaints and an investigation of bad housing in New York, showing that serious defects are widely prevalent, bring no new facts but merely emphasize an evil long recognized and equally long left without cure. Now, however, with a house-building boom on and growing rapidly, it is time to plan purposefully for reform.

Building control in the case of major structures is essentially different from control of small building. The present problem is that of the small house, whether single dwelling or apartment. It is in this field that shoddy construction with resulting rapid deterioration and loss of value adds to the evils of poor planning and the associated factors that often make the new housing districts of large cities little better than foredoomed slums. And these same factors contribute heavily toward making houses too costly for most people to own.

Dwellinghouse construction in volume is largely done by speculative builders, who sell to the owner; the owner therefore cannot protect himself against inferior building, since he appears on the scene only when the house is completed and when its weaknesses are covered up. Financing institutions that furnish the money for such building operations are less interested in the excellence of what is built than they are in making sure of a sufficient margin between their loan and the probable market price of the house. Building laws are notoriously inadequate to protect, and despite the fullest feasible municipal inspection the profit-seeking motive of the speculative operator is under little restraint.

Among the correctives for the situation that have been proposed are: (1) stiffer building laws and inspection, (2) supervision by the institutions that provide the money which finances housing, (3) the licensing of builders, (4) certification of building quality by an inspecting agency and public record of the certificate of grade or quailty, and lastly, (5) mandatory supervision and certification of every building operation by a competent architect or engineer.

The principal hope lies in the fourth and fifth proposals, which depend on assured competence of supervision. For final cure of the evils of bad house-building and assurance of quality to the home owner, it may be taken for granted that certification and public record thereof in some form is the ultimate hope. Both of the two proposals mentioned involve such certification, though the means they employ for the purpose are different. Simplest and most effective for immediate application is the requirement that all building must be supervised and certified as to its adequacy by a registered architect or a licensed professional engineer of structural competence. It was this proposal, made a dozen years ago by a committee of structural safety, that has received the support of the mayor of New York City. As the most practical, direct and effective means of dealing with bad building, it should find early adoption.

Even more direct promise lies in a recently developed plan that combines in an interesting way the strength of technical planning and supervision with enforcement by financial control and public record. If put in practice successfully it should prove of countrywide influence and should mark the biggest step in progress toward correction of bad building that has ever been recorded. It is a plan worked out within the Home Loan Bank Board to provide "families of modest means the first protection they have ever had against poor housing," according to a recent issue of the Federal Home Loan Bank Review.

The essentials of the plan are the provision of competent technical service in designing and in supervision by specially formed local architectural groups at a cost within reach of everyone, the low cost to be realized through systematization of the technical service in a way not possible were the individual dealing with the architect direct. Experimental trial of the plan has shown that the service can be rendered for 1½ to 2 per cent of the construction cost, which is so far below what has heretofore been possible that it may be said to eliminate the burden of the architectural charge completely.

The nearly eight years of intermission in housebuilding have created a tremendous problem in the way of rehousing the nation. Large deficits in housing exist everywhere and must soon be made up. The current year's building record, substantially twice that of 1935, is an index of the rate at which a new housebuilding boom is preparing. To protect home standards, to safeguard the homeowner's savings, and to stabilize property investment everywhere, bad building must be controlled and ultimately eliminated. The service plan merits immediate trial on a full-sized basis to show how effectively it can fulfil this purpose.

Toward a Policy

MUCH ENCOURAGEMENT can be found in the conferences begun last week at the White House through which it is hoped to work out an agreement for cooperation between the electric utilities of the southeastern states and the Tennessee Valley Authority for pooling their power resources and cooperating in developing their markets. From the early days of the TVA, Arthur Morgan has favored cooperation with the private utilities if it could be brought about on satisfactory terms, but he has been opposed, even from within his own house, by those who favored fighting the utilities and competing for their markets. The President, by calling the White House conference, has sided with Dr. Morgan and so has paved the way for what may develop into a definite national power policy, something that now is notably absent, despite the appointment of a power policy committee many months ago.

Competition between public and private power bodies by duplication is bound to be costly in the end. Experience has shown that the best way to cut rates is to cut unit production costs and increase the power demand. At present the power consumption of the country is higher than it was in the prosperous days of 1929, though industry is still below normal. As business returns to normal the southeastern states will need much more power; yet no new plants other than those of the TVA have been built in that region. The TVA could not reach out to absorb the growing market over a wide area without costly duplication, but by pooling its power with that of the local utilities under an agreement that will pass on to the consumer the full benefits of low-cost power production and distribution it can attain its objectives efficiently.

CURRENT NEWS

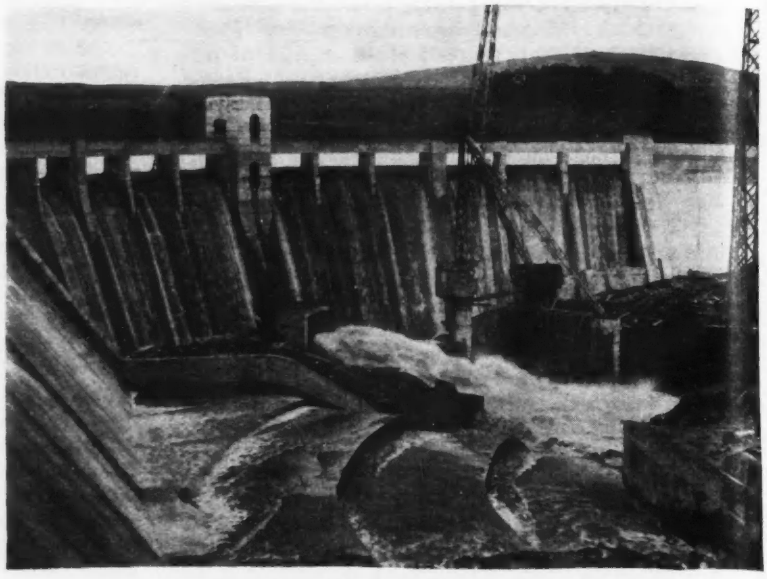
Joint Action Sought On Delaware River

The Delaware River Basin Commission will plan for flood control, conservation, navigation, and pollution abatement

OFFICIALS and representatives of four states met on Oct. 3 to define the purpose and policy of the Delaware River Basin Commission. It was voted that the commission, composed of representatives of New York, New Jersey, Pennsylvania, and Delaware, should supervise the apportionment and regulation of the flow of the Delaware River to protect communities and interests now sharing in its use. Thomas A. Logue, Pennsylvania secretary of internal affairs and chairman of the meeting, announced that proposed uniform measures to be presented to the legislatures of the several states would be drawn up at a meeting of the commission in December. These agreements would be drawn up with the assistance of the federal government.

The commission recommended the construction of a series of impounding dams on the upper Delaware as being essential for the development of the river. Other recommendations include: an aggressive policy of public acquisition of land for public recreational use; the elimination of all toll bridges; action to reconcile the fish and game legislation of the four states; and use of all lakes behind storage dams for recreational purposes.

A further recommendation of the commission called for the formation of an advisory committee composed of mayors and municipal officials in the Delaware River basin to cooperate with the commission in furthering the development of the basin, including such problems as flood control, conservation, navigation, recreation, and pollution abatement.



A BRITISH POWER PROJECT NEARS COMPLETION

The Galloway hydro-electric project will be in full operation in the early autumn. The project, for the development of the River Dee in Scotland, consists of five power stations and dams in a 40- mile length of the river. The project will develop 150,000 hp., has taken five years to complete, and has cost about $15,000,000.

Building Code Enacted for New York World's Fair

The New York City Board of Estimate and Apportionment has enacted a special building code to govern the erection and use of temporary buildings for the New York World's Fair of 1939. The new code supersedes the regular building code of the city of New York, liberalizing some of the provisions of that code in view of the fact that many of the buildings erected for the fair will be of a temporary nature. Permanent buildings on the fair site must conform to the regular city code. The code was prepared by the world's fair engineers in cooperation with the city building department. It will be administered by the corporation itself, which will have complete authority over the construction and use of all temporary structures at the fair and power to take court action in the case of violation.

The new code permits much greater freedom in the use of new materials, assembly, and types of construction than is permitted under the city code. Exit and standpipe requirements have been made more rigid because of the relaxation of structural and fire-resisting requirements and because of the large crowds expected to attend the fair. For the same reasons occupancies have been confined to ground floors. Fireproof structures are unlimited as to maximum area and height. Fire protected structures, a new class, are unlimited as to area, but cannot rise more than 225 ft. above the grade of an occupied floor. Other types of construction are restricted as to allowable height and floor space in proportion to their fire resistant qualities.

Work Starts on Vehicle Tunnel Under East River at New York

Work on the midtown vehicle tunnel from Manhattan to Queens in New York City started on Oct. 2. Ceremonies, at which President Roosevelt and Secretary Ickes spoke, marked the beginning of work on the $58,000,000 project.

The work now under way is the sinking of a construction shaft on the Queens side of the East River. Contract for this work was awarded to the G.A.M. Construction Co. of New York City on their low bid of $221,088.

Most Hospital Construction Financed by PWA

PWA has been responsible for 64 per cent of all hospital construction during the last three years. The PWA program has added 50,000 beds to the national hospital capacity, and has financed 466 hospital projects costing about $140,000,000.

This announcement was made in connection with the laying of the cornerstone, on Oct. 2, of the new medical building of the Jersey City, N. J., medical center, by President Roosevelt. The ceremony marked the halfway point in construction of a $7,500,000 program, financed by the Public Works Administration, which will make the Jersey City institution the third largest medical center in the United States.

Work on the steel framework of the medical building is well advanced. It is a 21-story fireproof building which will accommodate 600 patients. Excavation is now under way for the construction of a 10-story clinical building capable of serving about 800 out-patients daily. The 22-story Hudson County tuberculosis hospital has already been completed with PWA funds. In addition, one of the older buildings of the group is being improved and remodeled on the same project.

The Jersey City medical center is a joint activity of Hudson County and Jersey City. The county sponsored the construction of the tuberculosis hospital, and the other parts of the project were sponsored by the city.

Identical Bidding Found On Reclamation Projects

Two cases of identical bidding on materials to be purchased by the Bureau of Reclamation have been referred by Secretary Ickes to the Attorney General for such action as he may wish to take. In each of these cases, while the majority of bids were identical, at least one bidder submitted different and lower figures; both contracts were awarded to the low bidders.

One contract involved the supplying of copper cable and wire for the transmission line on the North Platte federal reclamation project. The Nehring Electrical Works of DeKalb, Ill., was awarded the contract on its bid at destination of $16,487. However, of the fourteen bids received by the Bureau of Reclamation on Aug. 20, seven offered goods at a delivered cost amounting to $16,501, and two additional bidders offered goods at identical delivered costs of $16,506.

The other contract was for steel tubing for the Grand Coulee Dam. The N. B. Austin Co. of Chicago was awarded this contract on its low bid at delivery of $19,742. Two other bids showed slight differences in shipping costs, but the remaining seven bids were identical, with a delivered cost of $22,013.

Conchas Dam Contract Awarded

The contract for the construction of Conchas Dam on the South Canadian River near Tucumcari, N. Mex., has been awarded to the contracting firm of Bent Bros., Inc. and the Griffith Co., both of Los Angeles. The contract was awarded by the U. S. Engineer Corps on a bid of $4,587,676.

Among other projects, Bent Bros., Inc. built the Morris Dam on the San Gabriel River for the city of Pasadena, Calif. The Griffith Co. is now completing Cajalco Dam, forming main impounding reservoir for the Metropolitan Water District of Southern California.

Way Cleared for New Highway Along the Florida Keys

The Interstate Commerce Commission has authorized the Florida East Coast Railway Co. to abandon its 125-mile line from Florida City to Key West and to sell the bridges and rights of way to the Overseas Toll and Bridge District, which proposes to construct a highway from Miami to Key West.

The section of railway which extends across the Keys was severely damaged by the hurricane of Septemer 1935. Although the concrete bridges which carry the line across many of the open water stretches were not damaged, those sections where the line was carried on embankment suffered severely. It is estimated that the cost of repairing the railway would have been $1,800,000 if wooden trestles were used. If a more permanent type of steel and concrete construction were used the cost would have amounted to $2,940,000. It was not considered that the potential traffic on the line was sufficient to justify the cost of the repairs.

E.C.P.D.'s Accrediting Program Well Advanced

Four Committee Reports to Annual Meeting Show Much Work Done—College Accrediting in Northeastern Region Nearly Completed

FULL REPORTS from the four main committees of the Engineers' Council for Professional Development were presented to the council's annual meeting, held at New York Oct. 6, with a full attendance which displayed keen interest in the work of the committees. So much work was represented in the reports, in fact, that plans were made to hold two meetings a year in place of the present single meeting. It is proposed to hold a spring meeting six months from now.

Dean R. L. Sackett, Pennsylvania State College, reported for the committee on student selection and guidance that tests have been made on entering students in a number of colleges and at the end of the first and second years to determine the correlation between the gradings of test examinations in English and mathematics at entrance with average grades of the same individuals in all subjects after one and two years in college. The committee believes that the method is of value in orientation.

The work of the past two years in carrying out the program of accrediting engineering curricula was summarized in the report of the committee on engineering schools, President Karl T. Compton, Massachusetts Institute of Technology, chairman. The majority of the engineering colleges in the New England and Middle Atlantic states have been visited by delegatory committees of the council, and a list of accredited curricula for the institutions so far covered is to be published shortly. The cost of this work was covered partly by inspection fees paid by the colleges visited and partly by a grant from the Engineering Foundation.

A comprehensive group of recommendations was presented by the committee on professional recognition, chairman C. N. Lauer, Philadelphia. Leading elements of the recommendations were that engineering societies adopt the E.C.P.D.'s "minimum definition of an engineer" as a minimum requirement for admission to membership; that the council and its participating bodies assist in extending and strengthening registration laws; and that registration be established as a minimum prerequisite for admission to different grades of membership in the national societies. Action on these recommendations was deferred to give opportunity for more consideration.

R. I. Rees, reporting for the committee on professional training, told of the preparation of a personal appraisal blank by the committee two years ago, revised and republished early this year; the initiation of a survey of educational facilities by the junior committee on professional training; the publication of "Suggestions to Junior Engineers"; and the preparation of a bibliography of outstanding books on technical subjects. This bibliography is to be published shortly. A program for definite assistance to junior engineers in their continuation training was organized in conjunction with the Providence Engineering Society.

Prof. Charles F. Scott was reelected chairman of the E.C.P.D. and R. I. Rees was elected to the new office of vice-chairman. H. H. Henline was elected secretary and C. E. Davies assistant secretary.

A.S.C.E. Asked to Reinstate Two Los Angeles Engineers

The board of supervisors of the Los Angeles Flood Control District has asked the American Society of Civil Engineers to reinstate two engineers who were expelled from the society in 1932 following revelation of the part they played in the exposure of alleged irregularities in connection with a contract for San Gabriel Dam No. 1 at the so-called Forks site. The engineers are B. F. Jakobsen and James H. Payne, both of Los Angeles. The supervisors' request is made in the form of a resolution that reads in part as follows:

> Be it resolved by the Board of Supervisors of the Los Angeles County Flood Control District that we publicly recognize the value to this district of the courageous engineering advice so freely given by said B. F. Jakobsen and James H. Payne, and that we memorialize the American Society of Civil Engineers to reconsider the expulsion from said society of these gentlemen in the light of the accomplishment which their action materially assisted in bringing to pass.

The controversy that had as an indirect result the expulsion of Messrs. Jakobsen and Payne was much involved. It centered around the contract of Fisher, Ross, MacDonald & Kahn for the construction of a high concrete dam at the Forks site in the San Gabriel canyon, a contract that was cancelled when extensive abutment excavation showed the site to be unsuitable. After a settlement had been made by the directors of the district (the county supervisors) with the contracting firm, charges were publicly made that the contractor had been overpaid, the charges being based, in part at least, on a survey of the excavation, which survey was made by Payne, who was engaged for the purpose by Mr. Jakobsen. These measurements purported to show that much less rock had been removed than was shown in the estimates of the district's chief engineer.

Following the public hearings of the case members of the American Society of Civil Engineers brought charges of unprofessional conduct against Jakobsen and Payne, based, it is understood, on the manner in which they had prepared and made public information injurious to the reputation of a fellow engineer. After investigating the charges, the board of direction of the society expelled Jakobsen and Payne "for cause."

Recent litigation between the flood-control district and the former contractor, and a resulting settlement, are believed to underlie the supervisors' resolution.

Good Progress Being Made On Rio Grande Canalization

The canalization program for the upper course of the Rio Grande between Texas and Mexico, which will cost a total of $5,000,000, is 70 per cent completed, according to announcement by officials of the International Boundary Commission. The construction program involves the straightening of the river channel between El Paso and Box Canyon. Federal funds have been set aside to guarantee uninterrupted work on the project during the next twelve months. Bids will be called for soon for construction of a diversion dam on the Rio Grande in the American Smelting and Refining Company community on the river below El Paso. The project will cost $1,000,000. The diversion dam, which will control the Rio Grande irrigation water flow for distribution in the El Paso and Juarez valleys below El Paso, also includes a one-mile section of diversion canal, which will connect the dam with the present Franklin Canal head gates. Federal funds have been appropriated for the work, which will start in 30 days after the construction bid is approved by federal officials. The dam will require eighteen months for completion.

Construction of the $2,500,000 Caballo Dam north of El Paso in New Mexico was started recently with the work expected to be finished in two years.

Bids on Detroit Housing Project Rejected by the PWA

Public Works Administrator Ickes has rejected all bids for construction of substructures and superstructure on the Brewster slum clearance and housing project in Detroit. The action was taken on the grounds that the bids greatly exceeded the estimates prepared by the Housing Division of the PWA.

The work involved consists of construction of the superstructure for eleven blocks of buildings and of foundations for buildings on six blocks. Clearance of the site has already been accomplished under another contract.

Bids for the work will be readvertised immediately by the Housing Division.

Big Generator Starts at Boulder Dam

Power was sent from Boulder Dam to Los Angeles on Oct. 7 when the first of the 115,000 hp. generators went into service. The first power generation at the dam occurred a month ago (*ENR*, Sept. 17, 1936, p. 423), Sept. 11, but only a small 3,500 hp. generator was put into service at that time, generating current for use at the dam and in Boulder City.

Ultimately there will be fifteen 115,000 hp. generators at Boulder Dam; four are now being installed. There will also be two 55,000 hp. generators, of which one is now being placed. When all units are installed the plant will have a capacity of 1,835,000 hp.

Use of WPA Forces For Snow Removal

Washington Correspondence

OFFICIALS of the WPA are considering whether it will be possible to use WPA forces in snow removal work this winter because municipalities are anxious to know in advance if they can reply on such help. In the past WPA officials have objected to any type of work that does not have a tangible product. Leaf-raking charges, for instances, have plagued them to such an extent that only projects of lasting usefulness have been regarded with favor.

Strong arguments are being brought, however, in an effort to show that snow removal should be undertaken this winter by WPA. Last winter in emergencies the WPA was forced into the work without having an opportunity to prepare for it in advance. Since the principal load on WPA is in the cities, men are concentrated at the points where the snow removal problem is greatest. This is a type of work that does not displace those already employed in the city forces.

Municipal authorities are interested because the recruiting of snow removal forces has been a serious problem. Large numbers of men cannot be assembled and organized overnight and under bad weather conditions. If WPA can be induced to take over the job, it is argued, planning will have been done in advance. Also, the WPA has trucks and other equipment to supplement the special equipment which the municipalities are in a position to supply from their own plant.

Gas-Tax Amendment Sought in Massachusetts

An initiative petition for an amendment to the Massachusetts constitution prohibiting diversion of motor vehicle revenues has been approved by the Secretary of the Commonwealth. Petition blanks for the signatures of voters are now being circulated by motorists and highway users organizations to obtain the necessary 25,000 signatures within the next 60 days as is required by Massachusetts law.

The proposed amendment provides that the proceeds of all gasoline taxes, registration fees, and other automotive levies, with the exception of money derived from the motor vehicle excise tax, shall be expended only for highway purposes, including highway policing and payment of principal and interest on outstanding highway bonds. The motor vehicle excise tax was exempted because this levy is imposed instead of a property tax.

Last year $15,000,000 of the $25,000,000 highway fund was diverted to uses unconnected with the highways. In the last five years $60,000,000 has been diverted.

Missouri, Minnesota, Colorado, and Maine already have adopted similar amendments. The Maine amendment was adopted at the recent state elections.

Small Dam Fails in Nebraska

The dam of the Northern Nebraska Power Co. on the Niobrara River, seven miles south of Spencer, Neb., went out without warning at 1 p.m. on Sept. 24. The failure was noted briefly in *ENR* Oct. 1, 1936, p. 486. The dam was an earth dike, 1,800 ft. long and about 18 ft. high, with a 400-ft. concrete spillway section.

The concrete spillway section failed suddenly and almost as a unit. Immediately afterwards the floodgates collapsed. It is believed that failure was caused by undermining of the soapstone rock on which the spillway section was built. An investigation has been started by the owners.

The dam was built in 1926 and has been operated by the Northern Nebraska Power Co. In the spring of 1935 a portion of the earth section of the dam failed. After a financial reorganization of the company, this dike was repaired at a cost of $70,000, and the plant has been operating since that time.

SOCIETY CALENDAR

NATIONAL SOCIETY OF PROFESSIONAL ENGINEERS, annual convention, Washington, D. C., Oct. 12-13.

AMERICAN SOCIETY OF CIVIL ENGINEERS, fall meeting, Pittsburgh, Pa., Oct. 13-17.

NATIONAL COUNCIL OF STATE BOARDS OF ENGINEERING EXAMINERS, annual meeting, Knoxville, Tenn., Oct. 19-21.

AMERICAN WELDING SOCIETY, annual meeting, Cleveland, Ohio, Oct. 19-23.

AMERICAN PUBLIC HEALTH ASSOCIATION, annual meeting, New Orleans, La., Oct. 20-23.

AMERICAN RAILWAY BRIDGE & BUILDING ASSOCIATION, annual meeting, Chicago, Ill., October 20-22.

AMERICAN INSTITUTE OF STEEL CONSTRUCTION, annual convention, White Sulphur Springs, W. Va., Oct. 21-23.

ASSOCIATED GENERAL CONTRACTORS, fall meeting of governing board, Washington, D. C., November 16-17.

HIGHWAY RESEARCH BOARD of the NATIONAL RESEARCH COUNCIL, annual meeting, Washington, D. C., Nov. 18-20.

AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS, annual meeting, San Francisco, Calif., December 7-10.

AMERICAN WATER WORKS ASSOCIATION, NEW YORK SECTION, fall meeting, Saratoga Springs, N. Y., October 15-16.

STRUCTURAL ENGINEERS ASSOCIATION OF CALIFORNIA, annual convention, Santa Maria, Cal., October 16-18.

AMERICAN WATER WORKS ASSOCIATION, NEW JERSEY SECTION; NEW JERSEY WATER WORKS ASSOCIATION; and the SOUTH JERSEY ASSOCIATION OF WATER SUPERINTENDENTS; joint meeting, Atlantic City, N. J., Oct. 23-24.

AMERICAN WATER WORKS ASSOCIATION, MISSOURI VALLEY SECTION, Kansas City, Mo., October 26-28.

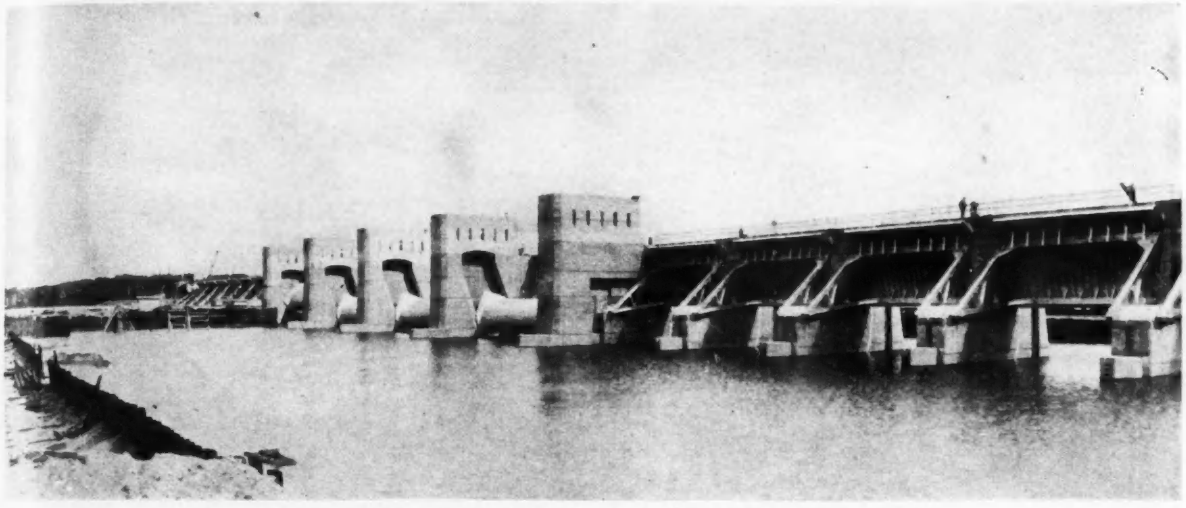
AMERICAN WATER WORKS ASSOCIATION, NORTH CAROLINA SECTION, Charlotte, N. C., October 26-28.

CENTRAL STATES SEWAGE WORKS ASSOCIATION, annual meeting, Indianapolis, Ind., Oct. 29-30.

AMERICAN WATER WORKS ASSOCIATION, VIRGINIA SECTION, Charlotteville, Va., October 29-31.

LICENSE EXAMINATIONS

NEW YORK STATE, examination for professional engineers and surveyors, Jan. 25-27, at Albany, Buffalo, New York City and Syracuse. Applications must be filed with the State Education Department at Albany before Nov. 1.



GIVING THE UPPER MISSISSIPPI A NINE-FOOT CHANNEL

Dam 16 of the 9-ft. channel canalization program for the upper Mississippi is nearing completion. This $2,053,130 dam, at Muscatine, Ia., consists of 15 tainter gates, 40 ft. long, and four roller gates, 80 ft. long; 1,700 ft. of ogee spillway, and several hundred feet of earth dam, bringing the total length to 3,940 ft. The dam is being built by the Central Engineering Co. of Davenport, Ia., under the supervision of the U. S. Engineers.

C. N. Railway Calls for Bids On 99-mi. Branch in Quebec

The Canadian National Ry. has called for bids on construction of the new branch line between Senneterre, on the C.N.R. transcontinental line, and Rouyn. (*ENR*, July 16, 1936, p. 100). The work involved is clearing and grading the right of way and construction of the substructure on two bridges.

One bridge crosses the narrows between Lake LeMoine and Lake Dubuisson; the other crosses the Kinojevis River, nine miles east of Rouyn. The waterways to be crossed are each about 1,000 ft. wide at high water.

Time limits are set so as to permit track laying to start from Senneterre early next summer and to be finished by the 15th of November, 1937. Track laying will be carried out by the railroad's own forces.

New York Subway Bids Opened

Bids for construction of the third section of the Sixth Avenue Subway, New York, extending from 33d to 40th St., were opened on Oct. 3. The Park Contracting Corp., a subsidiary of the Rosoff Subway Construction Co., was low with a bid of $7,834,606. Another Rosoff subsidiary is now building the subway section from 40th to 47th St.

A month ago, bids which had been asked for the construction of this section were thrown out when J. F. Cogan Co., the low bidder, was unable to qualify. The Cogan Co.'s bid was $6,492,626.

The other bids in the group just now opened were: George H. Flinn Corp., $8,182,545; Frederick L. Cranford, Inc., $8,193,567; Arthur A. Johnson-Necaro Co., Inc., $9,326,014; and the Carleton Co., Inc., $9,553,494.

Washington Highlights

By Paul Wooton

The War Department, on Sept. 28, approved an additional allotment of $1,600,000 for the continuation of construction work on the Bonneville Dam on the Columbia River in Oregon.

Field parties are actively at work in the three Louisiana floodways on what the Bureau of Agricultural Engineering describes as the greatest land appraisal job ever undertaken, in so far as acreage is concerned. More parties are to be put in the field as soon as they are organized.

By combining forces, advocates of the St. Lawrence waterway and of the Florida canal hope to secure the ratification of the treaty covering the former project and to secure an appropriation that will allow continuance of work on the latter project. Friends of the Florida project conferred in Washington September 28.

Since highway appropriations have been authorized through 1939, no federal-aid legislation will come before the next session of Congress which meets Jan. 5. It is expected that the report on uniform traffic laws being made jointly by the Bureau of Public Roads and the Highway Research Board will recommend legislation intended to increase highway safety.

The Interstate Commerce Commission has given formal approval to the abandonment of the ferry across San Francisco Bay between San Francisco and Alameda operated since 1887 by the Southern Pacific Railway. Similar action has already been taken by the California Railroad Commission. No date is set for the order to take effect, since the ferry will continue operation until the completion of the bridge now being constructed across San Francisco Bay.

Valverde Tunnel Completed On Colorado River Aqueduct

Completion of the Valverde tunnel on the Colorado River aqueduct was announced Sept. 30. Started in June, 1933, this 7¾-mi. tunnel is one of thirty-eight on the aqueduct system, fourteen of which have been entirely completed. Completion of work on the Valverde tunnel also completes the 20-mi. section of the main aqueduct west of the San Jacinto tunnel.

The Valverde tunnel is at the western terminus of the 242-mi. main section of the Colorado River aqueduct. Water from the tunnel will flow into the Cajalco Reservoir and will then be delivered to the cities of the water district through a 150-mi. distributing system. The completed aqueduct west of Mt. San Jacinto includes nearly twelve miles of conduit and siphons and the 6,200-ft. Bernasconi tunnel, completed in November, 1935, in addition to the Valverde tunnel.

Of the 92 miles of tunnels on the main aqueduct line, 86 miles have been excavated since December, 1932, and 57 miles have been lined with concrete. Including the tunnels on the distribution lines, there are 108 miles of tunnels on the Colorado River aqueduct system.

Brief News

FORMATION OF AN ENGINEER CORPS in the Cuban army to aid in planning rural development and road and bridge building throughout Cuba will take place shortly, it is announced. It is expected that the corps will make a detailed study of the Cuban soil, locate highways and bridges, and take general charge of all rural planning.

FOUR CEMENT SPECIFICATIONS have been drafted by a special governmental committee under the direction of P. H. Bates, of

NEWEST FEDERAL OFFICE BUILDING

The new $11,000,000 building in the left center was built for the Department of the Interior as a PWA project. It provides 700,000 square feet of new floor space for the department. The old building is to the right of the new one.

the Bureau of Standards, and submitted to the Federal Specifications Board for adoption. They cover standard portland, high-early-strength, moderate heat, and sulphate-resistant cements. All are characterized by a compression test alternate and the requirement of high fineness. Fineness is to be determined only by the turbidimeter method.

CONSTRUCTION OF A BRANCH of the Laredo-Mexico City highway will be started soon. It will meet the United States system of highways at Hidalgo, Texas, and run southwest from Reynosa, just across the border, to Monterey, 147 miles. The new road will shorten the automobile route from points in the eastern part of the United States to Mexico City by more than 200 miles. The first allotment of money for beginning the building of this highway has been made by the Mexican government.

WORK ON THE FIRST SPILLWAY SECTION at Pickwick dam was put under way about Oct. 1. Excavation on the north bank of the Tennessee River for the foundation of the power house started at about the same time. Two steel towers for a cableway across the river have been completed, and the cable will be strung within a short time. This cableway will transport the concrete across the river for power house construction. The navigation locks, located on the south bank of the river, are practically complete. The concrete building on the south bank which will house the lock machinery is about 85 per cent complete.

THE EGYPTIAN GOVERNMENT is about to ask for bids on the construction of roads connecting Cairo and Alexandria with the Suez Canal Zone. About $25,000,000 will be spent on these projects. One of the roads to be built will include two bridges over the Nile, which will cost about $1,250,000 each. Another road from Ismailia to Alexandria will cost about $7,500,000. These roads connect strategic military points in Egypt; they are being built in accordance with the recent Anglo-Egyptian treaty. Although the estimate for the construction of these roads is only $15,000,000, an appropriation of $25,000,000 is being made in order to have a large margin for incidental and unforeseen expenses.

Personals

HARRY JACOB has been appointed chief engineer of the Louisiana State Board of Engineers.

S. M. RUDDER, who has been engineer in charge of surveys and plans for the Missouri highway department, has been appointed assistant chief engineer of the department to succeed Carl W. Brown.

DAVID SHEPHERD, a member of the civil engineering firm of Campbell & Shepherd, of Toronto, Ont., has been appointed housing adviser to the Canadian National Employment Commission. Mr. Shepherd is a graduate of Edinburgh University.

MAJOR HERMAN H. POHL, Corps of Engineers, has been relieved from his assignment with the Fifth Engineers at Fort Belvoir, Va., and has been assigned to duty as assistant to the division engineer of the Ohio River Division at Cincinnati, Ohio.

ALBERT E. MACDONALD recently was appointed head of the department of civil engineering and architecture in the University of Manitoba at Winnipeg, Man. Professor MacDonald is a graduate of Dalhousie University and of Nova Scotia Technical College.

DAVID J. PEERY has resigned his position with the Missouri state highway department as bridge inspector on the Missouri River bridge near St. Charles, Mo., to accept a position as instructor in civil engineering at the Missouri School of Mines.

HENRY P. EVANS, JR., has resigned his position in the outlet work design section of the Denver office of the United States Bureau of Reclamation to accept an appointment as an associate in the civil engineering department at the University of Illinois.

WALLACE W. SANDERS has been appointed district engineer of the third district of the Kentucky state highway department. Mr. Sanders has been connected with the state highway department for twelve years, serving in various capacities. The offices of the third district are to be moved to Louisville.

ARTHUR S. TUTTLE has been appointed PWA project engineer for the new $58,000,000 Queens Midtown Tunnel, New York City, in addition to his duties as state PWA director for New York State. Mr. Tuttle has been head of the federal public works program in New York since PWA was established three years ago.

JOHN H. BATEMAN, engineer-manager of the Berrien County, Mich., road commission, has recently been appointed research professor of engineering and assistant director of the engineering experiment station at Louisiana State University. Professor Bateman will continue to serve the Berrien County road commission as consulting engineer.

Obituary

ROBERT A. MCCULLOUGH, a building contractor of Vancouver, B. C., died September 21 at the age of 76.

MICHAEL J. RYAN, for 35 years superintendent of streets at Swampscott, Mass., died there on September 27 at the age of 74.

JOHN D. WILKENS, who was among the designers of the Brooklyn, Williamsburg, and Manhattan bridges in New York City and who was connected for several years

with the bridge department of the New York Central Railroad, died in Dallas, Texas at the age of 81 on Oct. 3.

JOHN A. RUNNER, 64, who served as an engineer in the building of Cheesman dam in Colorado, died in Lovelock, Nev., on Sept. 5.

SIR BRODIE H. HENDERSON, senior partner in the consulting engineering firm of Livesey & Henderson and past president of the Institution of Civil Engineers, died in London on September 28 at the age of 67.

HERMAN ROSENTRETER, who, prior to his retirement in 1933, was division engineer of the Newark, N. J., water department, died in Morristown, N. J., on October 1 at the age of 70. Mr. Rosentreter had been with the water department for 26 years at the time of his retirement. Before serving with the water department, he had served with the Essex County Parks Commission, the Passaic Valley Flood Commission, and the Long Island Railroad.

JOHN CASSAN WAIT, 76, civil engineer and lawyer, died in Norwich, N. Y., on Oct. 4. Mr. Wait, after graduating in engineering from Cornell University and practicing for some years, took up the study of law, and devoted the larger part of his career to the subject of contract law, in which he became an authority of wide note. In his earlier years he had served in the construction of the New York canals and as assistant professor of engineering at Harvard University, but his chief work was his activity in the field of contract jurisprudence. His works in this field, including "The Law of Operations Preliminary to Construction", "Law of Contracts", and "Engineering and Architectural Jurisprudence", contributed to the development of equitable specification and contract practice. He also edited the "Car Builders Dictionary" and the "Calendar of Invention and Discovery."

CONSTRUCTION STATISTICS FOR THE WEEK

ENGINEERING construction awards total $48,334,000, eight per cent higher than the average week to date but under the three preceding weeks. The total this week is made up of private, $18,570,000; public, $29,764,000; federal, $4,920,000; state and municipal, $24,844,000. Corresponding values for a year ago are: total $44,700,000; private, $19,395,000; public, $25,305,000; federal, $6,234,000; state and municipal, $19,071,000.

Industrial building awards rose to $12,294,000 this week, sewerage awards are slightly up, bridge construction volume is higher, but other classes show a loss. The classified totals are: commercial buildings, $6,029,000; public buildings, $8,865,000; highways, $8,117,000; bridges, $2,196,000; sewerage, $1,327,000; waterworks, $648,000; earthwork, drainage, $4,722,000; unclassified, $4,136,000.

The larger projects for the week include: power plant improvements, Union Electric Light & Power Co., St. Louis, Mo., Illinois and Missouri, $10,000,000; plant and laboratory, Linde Air Products Co., Tonawanda, N. Y., $1,000,000; apartment, Madison Ave. and 72nd St., J. T. Smith, New York, N. Y., $1,000,000; Abraham Lincoln High School, Los Angeles, Calif., $635,000; superstructure, Smithfield Court Housing Project, Birmingham, Ala., $1,516,000; printing office warehouse, Treasury Department, Washington, D. C., $1,043,000; highways, Mississippi, $1,588,000; Nebraska, $588,000; building to house sludge and filter beds, Cleveland, Ohio, $534,000; rehabilitating drainage system, East Side Levee and Sanitary District, East St. Louis, Ill., $3,562,000; airport, Buffalo, $1,304,000; relocating Baltimore and Ohio RR., Bolivar Reservation, Zanesville Dam, Ohio, $528,000.

New capital consists of $4,741,000 in state and municipal bond sales and $100,000 in corporate security issues.

CONTRACTS

(Thousands of Dollars)

	Weekly Average Oct. 1935	Prev. 4 Weeks	Week Oct. 8 1936
Federal Government	$6,563	$7,104	$4,920
State and Municipal	17,907	23,433	24,844
Total public	$24,470	$30,537	$29,764
Total private	12,056	23,641	18,570
Week's total	$36,526	$54,178	$48,334
Cumulative to date: 1935	$1,090,391,000	1936	$1,843,063,000

NEW PRODUCTIVE CAPITAL

(Thousands of Dollars)

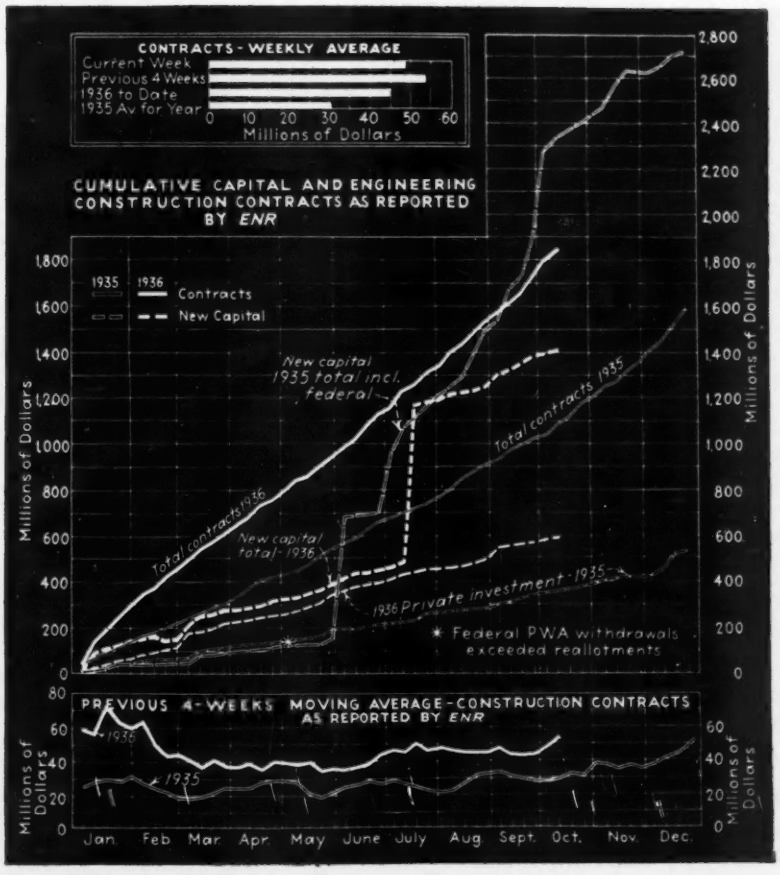
1936	Week Oct. 8	Cumulative
State and municipal	$4,741	$392,033
PWA non-federal	....	91,084
RFC loans	....	26,663
Corporate issues	100	204,009
PWA loans, Private	....	—1,665*
Total Non-Federal	$4,841	$712,124
Federal	....	691,237
Total new capital	$4,841	$1,403,361
Cumulative to date: 1935	$2,334,278,000	1936	$1,403,361,000

*Bond sales in this classification exceed reallotments during current year.

Note: These figures include private bonds, and stocks sold for productive purposes; state and municipal bonds for construction; PWA loans and grants to states and municipalities. RFC direct purchase of bonds for "self liquidating" projects, and 25 per cent of WPA construction appropriations.

INDEX NUMBER

ENR Cost	1913 = 100	1926 = 100	ENR Volume	1913 = 100	1926 = 100
Oct., 1936	211.50	101.67	Sept., 1936	.200	88
Sept., 1936	208.10	100.08	Aug., 1936	.199	87
Oct., 1935	195.10	93.82	Sept., 1935	.140	61
1935 (Av.)	195.22	93.84	1935 (Av.)	..135	58
1934 (Av.)	198.10	95.23	1934 (Av.)	..114	50
1933 (Av.)	170.18	81.80	1933 (Av.)	..102	47



Current Construction Unit Prices

Quabbin Earth Dam, Massachusetts

QUABBIN DAM, the last major structure in the development of the Quabbin Reservoir project on the Swift and Ware Rivers in central Massachusetts, was placed under contract late in July, 1936, by the Metropolitan District Water Supply Commission. The dam, located on Swift River near Enfield, Mass., is a hydraulic fill embankment containing 3,850,000 cu. yd. of earth and 150,000 cu. yd. of riprap facing on the upstream slopes. Principal dimensions are: crest length, 2,650 ft.; maximum height, 165 ft.; base width, about 720 ft. The dam will have an impervious central core 100 ft. wide at the base, 14 ft. wide at the top, held in place by pervious shoulders. A spillway channel will be cut out of solid rock, and spillway control consists of only a masonry weir. The dam will carry a concrete highway, crossing the spillway channel on a concrete bridge. Under previous contracts a concrete corewall to rock, diversion tunnel and upstream and downstream cofferdams have been built. The Commission outlines five suggested plans of construction in an addenda to the specifications, all based on dry haul to hog boxes and thence sluicing of the material to place. Frank E. Winsor is chief engineer for the Commission.

Following are unit prices of the three low bidders. (A) Benjamin Foster Co., Philadelphia (contract) $2,317,445; (B) Coleman Groves Co., Boston, $2,639,100; (C) Martin Wunderlich Co., Jefferson City, Mo., $2,662,900. Nine bids were submitted, the highest being $4,555,800.

QUABBIN DAM, SWIFT RIVER, MASSACHUSETTS

Item	A	B	C
1. Clearing—60 acre	$150.00	$65.00	$150.00
2. Grubbing—5 acre	150.00	140.00	150.00
3. Earth excavation—250,000 cu.yd	.45	.22	.31
4. Rock excavation, loose—1,000 cu.yd	6.50	6.00	6.00
5. Rock excavation—125,000 cu.yd	1.30	1.85	1.00
6. Pervious emb., consolidated—25,000 cu.yd	.65	.38	.40
7. Impervious emb., rolled—20,000 cu.yd	.70	.38	.45
8. Hydraulic emb.—3,550,000 cu.yd	.32	.38	.42
9. Misc. embankment—100,000 cu.yd	.35	.38	.25
10. Select course emb.—150,000 cu.yd	.60	.38	.50
11. Crushed stone, gravel—30,000 cu.yd	1.30	.55	1.00
12. Riprap, light—125,000 cu.yd	.80	.75	1.00
13. Riprap, heavy—25,000 cu.yd	1.50	2.50	1.75
14. Paving—3,000 cu.yd	4.00	5.50	10.00
15. Soil dressing—70,000 cu.yd	.85	1.40	.70
16. Seed & grass—50 acre	130.00	500.00	200.00
17. Road subbase, gravel—9,000 cu.yd	1.25	1.10	1.00
18. Stone base, crushed—2,800 cu.yd	6.00	4.50	3.00
19. Stone surface, broken—4,200 ton	3.70	2.75	4.00
20. Bituminous material—60,000 gal	.15	.15	.17
21. Cable guard rail—2,000 lin.ft	1.00	.80	1.00
22. Boulder guard rail—600 cu.yd	3.00	3.50	5.00
23. Stone walls—3,000 cu.yd	7.00	12.00	10.00
24. Drill holes, rock, masonry—5,500 lin.ft	1.30	1.00	1.00
25. Furnish, place grout—1,500 cu.yd	11.00	22.00	12.00
26. Prepare rock surface—2,500 sq.yd	.65	1.00	.50
27. Portland cement—23,000 bbl	3.40	3.00	2.50
28. Concrete, cl. A—3,500 cu.yd	11.00	14.50	14.00
29. Concrete, cl. B—4,500 cu.yd	17.00	17.50	17.00
30. Concrete, cl. C—1,400 cu.yd	17.00	16.00	17.00
31. Reinforcing steel—150,000 lb	.05	.06	.05
32. Stone masonry, cl. A—330 cu.yd	21.00	165.00	160.00
33. Stone masonry, cl. B—600 cu.yd	18.00	55.00	100.00
34. Brick masonry—70 cu.yd	25.00	55.00	80.00
35. Steel pipe—350,000 lb	.10	.10	.08
36. Mortar lining, pipe—17,000 sq.ft	.40	.50	.25
37. Misc. steel, iron—60,000 lb	.10	.15	.15
38. Galvanizing—30,000 lb	.13	.10	.04
39. Misc. cast iron—275,000 lb	.10	.12	.09
40. Misc. non-ferrous metal—3,000 lb	.50	1.00	.60
41. Set & store metal work—200,000 lb	.05	.05	.03
42. Misc. lumber—20,000 F.B.M	80.00	100.00	150.00
43. Tile pipe, 12 in.—4,500 lin.ft	1.00	1.00	1.50
44. Tile pipe, 18 in.—2,000 lin.ft	2.00	2.00	2.00
45. Tile pipe, 24 in.—100 lin.ft	4.00	3.00	3.00
46. Sample wells—2,000 lin.ft	1.30	1.00	1.50
47. Stream control—total	13,000.00	2,000.00	5,000.00
48. Plug stream control tunnel—total	6,000.00	1,000.00	2,000.00
49. Intake structure—total	4,000.00	5,000.00	5,000.00
50. Maintain engineer offices—20 months	100.00	50.00	200.00
51. Clean up—total	5,000.00	1,000.00	2,000.00

Dam No. 22, Mississippi River

A DAM to consist of ten 60x25-ft. tainter gates, three 100x25-ft. roller gates, 1,600 ft. of submersible earth dike and a 200-ft. storage yard will be built on the Mississippi River 1.4 miles below Saverton, Mo. The structure, known as Dam No. 22, will be under the direction of the Rock Island District, U. S. Engineer Dept.

Bids were opened Aug. 18, 1936, and contract was awarded shortly afterwards to the low bidder. Seven bids were received, and the engineer's estimate of $2,306,269 was higher than the three low bids. Unit prices of the three low bidders follow: (A) The Massman Const. Co., Kansas City, Mo., (contract) $2,111,052; (B) Dravo Contr. Co., Pittsburgh, Pa., $2,169,415; (C) Central Engineering Co., Davenport, Ia., $2,214,497.

DAM NO. 22, MISSISSIPPI RIVER

Item	A	B	C
1. Cofferdam—1,078 l.f	$231.50	$200.00	$266.00
2. Clearing—lump sum	1,000	1,000	1,500
3. Excav., common—50,000 cu.yd	.50	.30	.45
4. Excav., dredged—23,000 cu.yd	.80	.50	.35
5. Excav., topsoil—1,000 cu.yd	.40	.50	.40
6. Excav., rock—9,000 cu.yd	2.50	5.00	4.25
7. Rock channeling—10,600 sq.ft	1.25	1.00	1.10
8. Grout holes, first 10 ft.—800 l.f	.60	.80	.60
9. Grout holes, second 10 ft.—400 l.f	.75	1.00	.70
10. Pressure grouting, cement—200 bbl	6.00	10.00	7.50
11. Embankment, dike—74,000 cu.yd	.20	.30	.30
12. Piling, concrete—1,950 l.f	3.50	3.00	3.00
13. Piling, MZ 38 steel sheet—12,000 sq.ft	1.50	1.60	1.46
14. Piling, I 23 steel sheet—136,000 sq.ft	.50	.60	.75
15. Timber, oak—3 M fbm	250.00	250.00	275.00
16. Timber, softwood plain & creo.—45 M fbm	189.00	225.00	185.00
17. Stone, derrick—5,400 cu.yd	3.00	4.00	3.33
18. Stone, one & two man—15,500 cu.yd	3.00	4.00	2.83
19. Stone, riprap—7,500 cu.yd	3.00	4.00	2.83
20. Stone, crushed or gravel—8,500 cu.yd	2.50	3.70	2.55
21. Concrete, class A—750 cu.yd	24.00	30.00	22.50
22. Concrete, class B—38,500 cu.yd	12.40	15.00	14.11
23. Concrete, slushing—850 cu.yd	12.00	15.00	9.75
24. Steel, service bridge—1,530,000 lb	.07	.07	.07
25. Steel, tainter gate mov. parts—1,870,000 lb	.10	.09	.10
26. Steel, roller gate mov. parts—1,150,000 lb	.10	.09	.10
27. Steel, misc. structural—1,175,000 lb	.08	.07	.07
28. Steel, struc. manganese—360,000 lb	.09	.08	.09
29. Steel, struc. chromium—73,000 lb	.11	.11	.10
30. Steel, reinf.—1,100,000 lb	.05	.06	.05
31. Steel, corrosion-resisting—20,000 lb	.60	.60	.60
32. Forgings, steel—420,000 lb	.12	.12	.10
33. Forgings, nickel steel—3,900 lb	.20	.30	.20
34. Forgings, chromium steel—29,500 lb	.16	.15	.14
35. Castings, steel grades 1 to 3—330,000 lb	.16	.14	.15
36. Castings, alloy steel—43,500 lb	.18	.15	.18
37. Castings, iron—250 lb	.10	.32	.10
38. Bronze—10,200 lb	.50	.44	.50
39. Pipe, 4-in. w.i. std. blk.—19 ft	3.00	4.00	3.50
40. Pipe, 3-in. w.i. std. blk.—350 ft	2.00	2.00	2.50
41. Pipe, ¾-in. std. blk.—525 ft	.75	.60	1.25
42. Tubing, 1-in. copper—625 lb	1.50	1.00	1.10
43. Fence—3,015 l.f	.45	.60	.35
44. Conduit, 4-in. fibre—120 ft	.80	1.00	.85
45. Conduit, 3-in. fibre—625 ft	.60	1.00	.65
46. Conduit, 3-in. rigid metal—140 ft	2.65	2.00	1.70
47. Conduit, 2-in. rigid metal—10 ft	2.50	1.50	1.00
48. Conduit, 1½-in. rigid metal—625 ft	1.35	1.20	.80
49. Conduit, 1¼-in. rigid metal—20 ft	1.35	1.50	.70
50. Conduit, 1-in. rigid metal—200 ft	1.25	1.00	.65
51. Conduit, ¾-in. rigid metal—185 ft	1.00	.80	.60
52. Electrical grounding system—total	500.00	200.00	450.00
53. Crane track, 80-lb. rail—1,400 trk.ft	5.00	4.25	4.00
54. Handrail, steel pipe—1,484 ft	3.00	3.00	2.00
55. Steel floor grating—205 sq.ft	2.00	1.40	1.00
56. Molded rubber seals—6,500 lb	1.00	1.00	1.00
57. Roller gate oper. mach.—3 each	25,000	23,500	24,000
58. Roller gate mach. spare parts—1 lot	10,000	9,200	10,000
59. Roller gate oper. house—4 each	5,000	5,000	5,000
60. Tainter gate oper. mach. for non-submergible gates—9 each	15,500	12,500	15,000
61. Tainter gate oper. mach. for submergible gates—1 each	17,500	16,800	17,000
62. Tainter gate spare parts—1 lot	500.00	300.00	500.00
63. Tainter gate mach. spare parts—1 lot	10,000	8,000	10,000
64. Tainter gate heater—6 each	600.00	450.00	600.00
65. Tainter gate heater spare parts—1 lot	400.00	400.00	400.00
66. Roller gate heater—3 each	3,200	3,100	3,200
67. Roller gate heater spare parts—1 lot	775.00	900.00	775.00
68. Locomotive crane, install only—total	2,000	2,000	1,500
69. Storage yard truck—14 each	300.00	275.00	300.00
70. Service bridge truck—2 each	350.00	425.00	350.00